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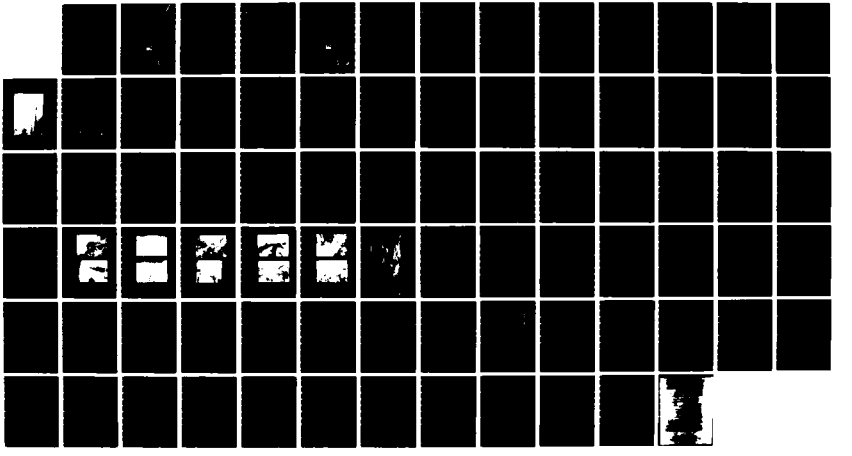

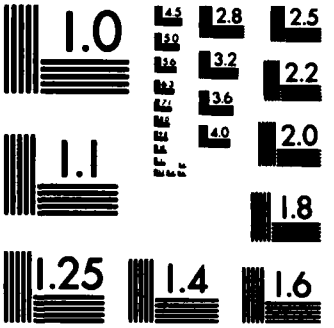

CONNECTICUT RIVER BASIN
SIMSBURY, CONNECTICUT

TULLER RESERVOIR DAM
CT 00275

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER, 1980

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
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4. TITLE (and Subtitle) Tuller Reservoir Dam NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS		5. TYPE OF REPORT & PERIOD COVERED INSPECTION REPORT 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U.S. ARMY CORPS OF ENGINEERS NEW ENGLAND DIVISION		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS DEPT. OF THE ARMY, CORPS OF ENGINEERS NEW ENGLAND DIVISION, NEDED 424 TRAPELO ROAD, WALTHAM, MA. 02254		12. REPORT DATE September 1980 13. NUMBER OF PAGES 55
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16. DISTRIBUTION STATEMENT (of this Report)

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17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Conn. River Basin
Simsbury, Conn.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The Tuller Reservoir Dam is an earth embankment built about 1900 and impounding Hop Brook in Simsbury, Conn. The dam is approx. 240 ft. in length (not including the spillway)., 7 ft. wide at the top, 45.5 ft. in height and has a maximum impoundment capacity of 175 acre-feet. In accordance with the Army Corps of Engineers Guidelines, Tuller Reservoir Dam is classified as a significant hazard, intermediate size dam. The test flood for Tuller Reservoir Dam is selected as equivalent to the PMF. Based upon the visual inspection at the site and past performance of the dam, the project is judged to be in poor condition.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:

NEDED

DEC 19 1980

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Tuller Reservoir Dam (CT-00275) Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, Village Water Company, Simsbury, Conn.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely,

WILLIAM E. HODGSON, JR.
Colonel, Corps of Engineers
Acting Division Engineer

Incl
As stated

CONNECTICUT RIVER BASIN

SIMSBURY, CONNECTICUT

TULLER RESERVOIR DAM

CT 00275

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASS. 02154

SEPTEMBER, 1980

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	TULLER RESERVOIR DAM
Inventory Number:	CT 00275
State Located:	CONNECTICUT
County Located:	HARTFORD
Town Located:	SIMSBURY
Stream:	HOP BROOK
Owner:	VILLAGE WATER COMPANY
Date of Inspection:	MAY 9, 1980
Inspection team:	PETER M. HEYNEN, P.E.
	MIRON PETROVSKY
	JAY. A. COSTELLO
	MURALI ATLURU, P.E.
	JEFFREY O. BORNE

The Tuller Reservoir Dam is an earth embankment built about 1900 and impounding Hop Brook in Simsbury, Connecticut. The dam is approximately 240 feet in length (not including spillway), seven (7) feet wide at the top, 45.5 feet in height and has a maximum impoundment capacity of 175 acre-feet. The spillway is a concrete lined channel cut into bedrock at the right end of the dam. It consists of an 18 foot long, trapezoidal shaped concrete weir, a concrete training wall on the left side and a vertical rock cut on the right side. A gatehouse with a concrete foundation and brick superstructure is located at the central portion of the upstream slope. The low-level outlet is an 18 inch cast iron pipe extending through the gatehouse to the upstream slope. Three intake pipes allow water into a chamber in the gatehouse foundation which drains through the low-level outlet. The slopes and top of dam are covered with grass, dense brush, and tree saplings. A footpath extends along the top of the dam for the entire length.

In accordance with the Army Corps of Engineers Guidelines, Tuller Reservoir Dam is classified as a significant hazard, intermediate size dam. The test flood range is from one-half the Probable Maximum Flood (1/2 PMF) to the Probable Maximum Flood (PMF). The test flood for Tuller Reservoir Dam is selected as equivalent to the PMF. Peak inflow to the reservoir at the test flood is 1750 cubic feet per second (cfs) and peak outflow is 1630 cfs with the dam overtopped 1.4 feet. The spillway capacity with the lake level to the top of the dam is 415 cfs, which is 25% of the routed test flood outflow.

Based upon the visual inspection at the site and past performance of the dam, the project is judged to be in poor condition. There are areas requiring repair, maintenance and monitoring such as concrete deterioration, slope erosion, brush on the slopes and a seep on the downstream slope.

It is recommended that the owner retain the services of a hydraulic/hydrologic engineer to analyze in more detail the adequacy of the existing project discharge and overtopping potential. Other items of importance are inspection of the low-level outlet, evaluation of a depression area at the top of the dam, origin of seepage on the downstream slope, concrete repair, slope erosion and regrading of the top of the dam. Recommendations should be made by the engineer and implemented by the owner.

The above recommendations and further remedial measures which are discussed in Section 7, should be instituted within one (1) year, of the owner's receipt of this report.

Peter M. Heynen, P.E.
Project Manager - Geotechnical
Cahn Engineers, Inc.

STATE OF CONNECTICUT
PETER M. HEYNEN
NO. 9618
REGISTERED
PROFESSIONAL ENGINEER

C. Michael Horton, P.E.
Department Head
Cahn Engineers, Inc.

STATE OF CONNECTICUT
CLARENCE M. HORTON
No. 8433
REGISTERED
PROFESSIONAL ENGINEER

This Phase I Inspection Report on Tuller Reservoir Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

RICHARD DIBUONO, MEMBER
Water Control Branch
Engineering Division

ARAMAST MAHTESIAN, CHAIRMAN
Geotechnical Engineering Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

The information contained in this report is based on the limited investigation described above and is not warranted to indicate the actual condition of the dam. The integrity of the dam can only be determined by a means of a monitoring program and/or a detailed physical investigation. The accuracy of available data is assumed where not in obvious conflict with facts observable during the visual inspection.

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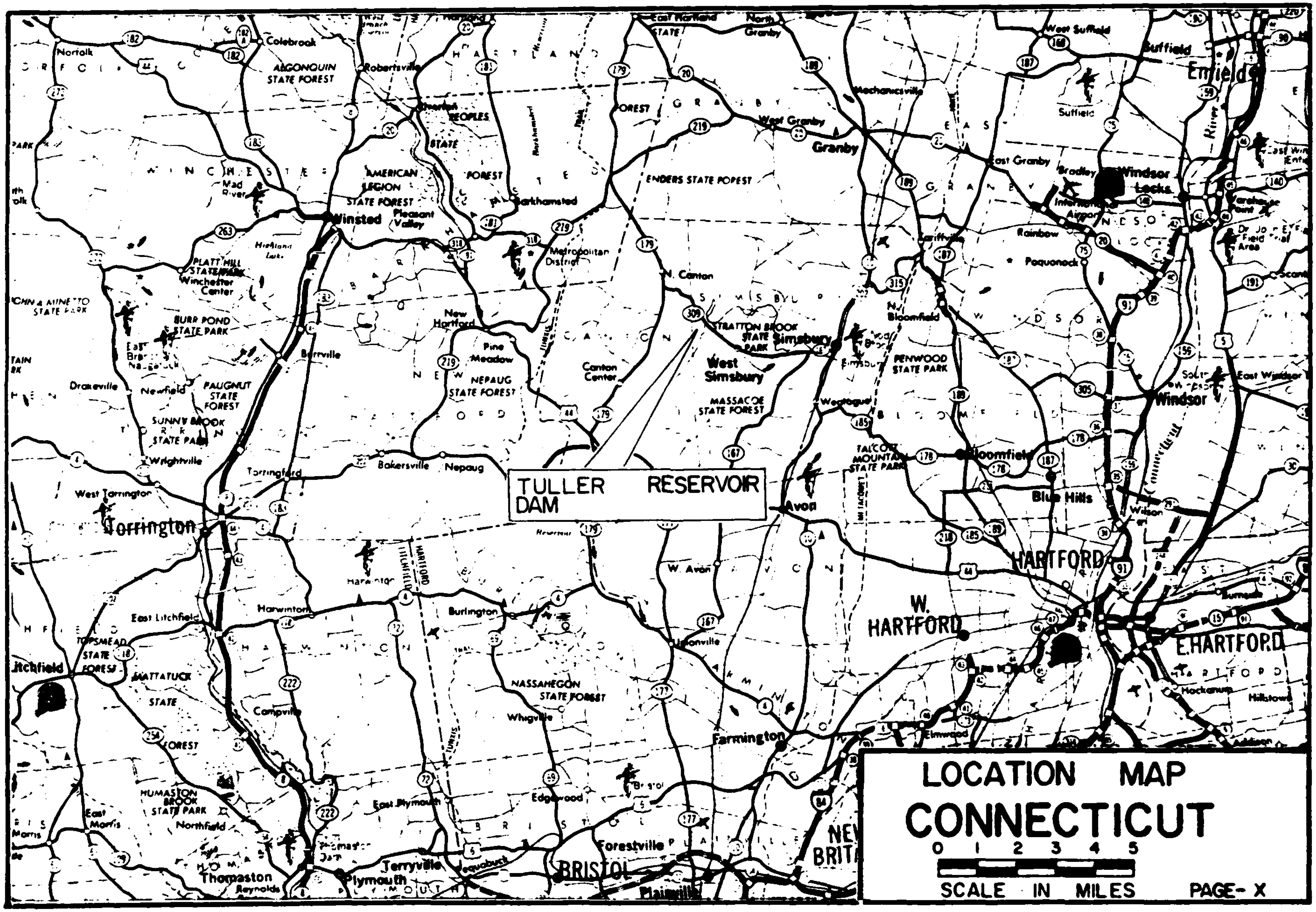
APPENDICES

OVERVIEW PHOTO
(May, 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Tuller Reservoir Dam	Simsbury	DATE Sept. 1980
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	INSPECTION OF NON-FED DAMS	Hop Brook	CONNECTICUT	CE #27 785 KE PAGE ix



TULLER RESERVOIR DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
PAGE- X
Torrington
Winsted
BRISTOL
HARTFORD
W. HARTFORD
E.HARTFORD
Windsor Locks
Windsor
Simsbury
West Simsbury
Avon
Farmington
Granby
Bloomfield
Blue Hills
Plymouth
Terryville
Thomaston
Forestville
Burlington
Harwinton
Canton Center
Colebrook
Norfolk
Suffield
ALGONQUIN STATE FOREST
AMERICAN LEGION STATE FOREST
NEPAUG STATE FOREST
NASSAHEGON STATE FOREST
MASSACOE STATE FOREST
PENWOOD STATE PARK
ENDERS STATE FOREST

PHASE I INSPECTION REPORT

TULLER RESERVOIR DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 14, 1980 from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0052 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report passes judgement only on those factors of safety and stability which can be determined by a visual surface examination. The inspection is to identify those visually apparent features of the dam which evidence the need for corrective action and/or further study and investigation.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on Hop Brook (Connecticut River Basin) in a rural area of the Town of Simsbury, County of Hartford, State of Connecticut. The dam is shown on Tarrifville USGS Quadrangle Map having coordinates latitude N41°53.0' and longitude W72°52.1'.

b. Description of Dam and Appurtenances - As shown on Sheet B-1, the dam is an earth embankment which is 45.5 in height and has a total length of 240 feet, including a 30 foot section at the left end which is less than 10 feet in height. There is a concrete lined spillway at the right end of the dam. This spillway length is not included in the total length of dam given above. All elevations are based on an assumed datum (spillway crest = 100.0) and are not N.G.V.D. elevations.

The dam is probably founded on rock and is 7 to 8 feet in width at the top, which at elevation 103.8, is 3.8 feet above the spillway crest. Low eroded areas, caused by a footpath extending the length of the dam, make the top of the embankment quite irregular. A concrete wall is exposed at the right end of the dam and extends from the left spillway training wall to the gatehouse, flush with the top of the embankment (See Sheet B-1, Photos 2 and 7).

The upstream slope is inclined at 2.5± (horizontal) to 1 (vertical) and is entirely stabilized with grasses and weeds above elevation 102.0. Below this elevation there is hand-laid riprap extending the length of the dam. The downstream slope is also covered with grasses and weeds with some small saplings. There is a 7-8 foot wide berm approximately halfway up the slope and extending the length of the dam. This berm runs on a slight downward slant from elevation 91± at the spillway wall to elevation 84± at the left end (See Sheet B-1). The downstream slope is inclined at 2 horizontal to 1 vertical above the berm and 3 horizontal to 1 vertical below the berm. There is a gravel service road which runs along the left abutment of the dam to the shore of the reservoir.

The spillway consists of a 160 foot long concrete lined discharge channel which runs along the right abutment with a concrete weir at the upstream end. The weir has a trapezoidal cross-section (2' wide at the crest and 4 feet wide at the base), a crest elevation of 100.0 and is 18 feet in length. A concrete training wall extends the length of the left side of the discharge channel and is 5 feet above the weir crest and 6-9 feet above the channel. The entire spillway is founded on bedrock, which also forms the left spillway wall (See Sheet B-1, Photos 5,6,7 and overview).

The gatehouse is a 14 foot square structure with a concrete foundation and a brick superstructure, and is located at the central portion of the embankment. As reported by the owner, the low-level outlet is an 18 inch cast iron pipe extending through the embankment from the upstream slope. This pipe outlets to a small pool at the toe of the dam (invert el. 58.3) at the end of the

spillway discharge channel (See Sheet B-1). There are 3 other inlet pipes (valves located in the chamber in the gatehouse) which extend from the gatehouse to the upstream slope. These inlets are situated so that there are two pipes (one at the right and left side of gatehouse) approximately 22 feet below the spillway crest and one pipe 6 feet below spillway crest. The two lower pipes are reported to be approximately 16 inches in diameter and the upper pipe 6 inches in diameter. All three pipes inlet to a chamber in the gatehouse foundation which can be drained through the low-level outlet. An abandoned 12 inch supply line to the filter plant is closed at the gatehouse with a section removed just before the filterplant.

The inlet structure is of concrete construction and unknown dimensions, located on the upstream slope. This structure has grates and is used only for the lower 16 inch inlet pipes. The 6 inch upper level pipe and low-level outlet have separate inlet structures of unknown construction.

c. Size Classification - INTERMEDIATE - The dam impounds 175 acre-feet of water with the reservoir level at the top of the dam, which at elevation 103.8, is 45.5 feet above the streambed of Hop Brook. According to Recommended Guidelines, a dam with this height is classified as intermediate in size.

d. Hazard Classification - SIGNIFICANT - If the dam were breached, there is potential for loss of less than a few lives at three houses, located 3300± feet downstream. These houses could be flooded by up to 1.5± feet of water and are located as follows:

- 1 house on Hop Brook south of Route 309 (opposite Hedgehog Lane) with first floor 11.5 feet above streambed.

- Two (2) houses located at southeast corner of Route 309 and Hedghog Lane.

e. Ownership - Village Water Company
Box 186
Simsbury, Conn.
Mr. Wentworth Hamilton (Superintendent)
(203) 658-6707

f. Operator - Mr. Gerald Bonadies
Village Water Company
(203) 658-6707

g. Purpose of Dam - The dam was constructed in the late 1920's to provide municipal water service to the Town of Simsbury. At the present time however the reservoir and its water treatment facilities are abandoned as a public water supply source.

h. Design and Construction History - The dam was constructed in the late 1920's. Plans for the dam construction could not be found although the owner reports that the name associated with the filter plant is "Singleton". In 1955, a portion of the spillway washed out and was replaced by the Village Water Company.

i. Normal Operational Procedures - The reservoir is no longer used for water supply, and has not been since 1970. At that time, the Village Water Company changed to groundwater wells for its municipal water supply.

At the present time, the upper level six (6) inch inlet pipe is open and provides the entire flow observed at the outlet pipe. The two lower inlet pipes are closed. When major storms (such as hurricanes) are anticipated, the low-level (18 inch) pipe is also opened.

The dam site is inspected daily by water company personnel. On occasion, when unauthorized recreational activity becomes too active or vandalism acute, the lower or mid-level valves are opened and the lake lowered 8-10 ft. to discourage trespassing.

A pipe which at one time conveyed water from the reservoir to the treatment works below the dam, has been terminated, and is no longer operational.

1.3 PERTINENT DATA

a. Drainage Area - 0.73 square miles of sparsely developed, rolling, wooded terrain in the Connecticut River Basin.

b. Discharge at Damsite - Normal discharge is over the spillway and through the low-level outlet pipe. Elevations listed are based on an assumed datum, spillway crest = 100.0.

1. Outlet works (conduits):	
18 inch cast iron low-level outlet at downstream invert el. 58.3:	60 cfs
2. Maximum flood at damsite:	Unknown
3. Ungated spillway capacity @ top of dam el. 103.8:	415 cfs
4. Ungated spillway capacity @ test flood el. 105.2:	670 cfs.

5. Gated spillway capacity @ normal pool el.:	N/A
6. Gated spillway capacity @ test flood el.:	N/A
7. Total spillway capacity @ test flood el. 105.2:	670 cfs
8. Total project discharge @ top of dam el. 103.8:	475 cfs.
9. Total project discharge @ test flood el. 105.2:	1630 cfs

c. Elevations - (Elevations are not NGVD. Elevations based on an assumed datum; spillway crest = 100.0).

1. Streambed at toe of dam:	58.3
2. Bottom of cutoff:	N/A
3. Maximum tailwater:	Unknown
4. Normal pool:	100.0
5. Full flood control pool:	N/A
6. Spillway crest (ungated):	100.00
7. Design surcharge (original design):	Unknown
8. Top of dam:	103.8
9. Test flood surcharge:	105.2

d. Reservoir (Length in feet)

1. Normal pool:	2,300 ft.
2. Flood control pool	N/A
3. Spillway crest pool	2,300 ft
4. Top of dam:	2,400 ft.
5. Test flood pool:	2,500 ft.

e. Storage (Acre-feet)

1. Normal pool:	134 acre-ft.

2.	Flood control pool:	N/A
3.	Spillway crest pool:	134 acre-ft.
4.	Top of dam:	175 acre-ft.
5.	Test flood pool:	190 acre-ft.
f.	Reservoir Surface (Acres)	
1.	Normal pool:	10 acres
2.	Flood control pool:	N/A
3.	Spillway crest pool:	10 acres
4.	Top of dam:	12 acres
5.	Test flood pool:	12 acres
g.	Dam	
1.	Type:	Earth embankment
2.	Length:	240 ft.
3.	Height:	45.5 ft.
4.	Top width:	7-8 ft.
5.	Side slopes:	2.5H to 1V Upstream 2H to 1V Downstream (above berm) 3H to 1V Downstream (below berm)
6.	Zoning:	N/A
7.	Impervious core:	Unknown
8.	Cutoff:	N/A
9.	Grout curtain:	N/A
10.	Other:	Berm along d/s slope
h.	Diversion and Regulating Tunnel	- N/A
i.	Spillway	
1.	Type:	Ungated concrete lined channel with concrete broad-crested trapezoidal weir

2. Length of weir:	18 ft.
3. Crest elevation:	100.0
4. Gates:	N/A
5. Upstream channel:	gravel, rock, gently sloping
6. Downstream channel:	concrete lined discharge channel along right abutment
7. General:	Spillway cut into bedrock, concrete left training wall, rock ledge along right side.

j. Regulating Outlets - (The only outlet is an 18 inch cast iron low-level outlet pipe).

1. Invert:	58.3 (d/s)
2. Size:	18"
3. Description:	Cast iron
4. Control Mechanism:	Hand operated floor stand at gatehouse.
5. Other:	3 inlet pipes valved in gatehouse. Two 16 inch lower level pipes at 22 feet below spillway crest. One 6 inch pipe at 6 feet below spillway crest.

SECTION 2: ENGINEERING DATA

2.1 DESIGN

The available drawings consist of a set of plans showing the water treatment facility at the toe of the dam. These include the location of the gate house and abandoned supply line and are available at the Village Water Company. No engineering values, assumptions, test results or calculations are available for the original design of the dam.

2.2 CONSTRUCTION

There is no data available for the original construction of the dam or subsequent repairs to the spillway after a flood in 1955.

2.3 OPERATION

The dam and reservoir are presently abandoned as a water supply facility and therefore, no formal operation data exists. Personnel from the Village Water Company visit the dam daily to make a visual inspection and patrol the grounds. Lake level readings are not formally recorded, but are visually noted during the daily visits.

2.4 EVALUATION

a. Availability - Existing data was provided by the State of Connecticut and verbally by Mr. Wentworth Hamilton, Superintendent, the Village Water Company. Mr. Hamilton made the project available for visual inspection.

b. Adequacy - The limited amount of engineering data available is inadequate to perform an in-depth assessment of the dam, therefore, the assessment of this dam must be based on visual inspection, hydraulic computations, hydrologic judgements and information provided verbally by water company personnel.

c. Validity - A comparison of the available information and visual observations reveals no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The condition of the dam is poor, based on the visual inspection performed on May 9, 1980. The inspection revealed items requiring various levels of maintenance, monitoring and repair. The reservoir was at elevation 100.2 with a small amount of water flowing over the spillway.

b. Dam

Top of Dam - The top of the dam is irregular and appears to be in poor condition. Several areas are eroded from a footpath which traverses its length (Photo 2). Erosion of the crest and upstream slope has revealed what appears to be a concrete cut-off wall at the right end of the dam (Photo 7). This wall is visible from the spillway to the gatehouse, and is flush with the top of the dam in this area. The top of the dam is vegetated with grasses and weeds (Photo 2). The rear of the gatehouse is located on the top of the dam. A shallow depression was observed in the vicinity of the gatehouse at the downstream side of the top of the dam (Photo 2).

Upstream Slope - The upstream slope of the dam is in fair condition. The hand-laid riprap protection is in good condition, however, weeds and small shrubs are growing between the stones. A large area of erosion was noted at the right end of the dam near the concrete spillway training wall. There is no slope protection in this area (Photo 7). The existing slope protection is weeds and grasses with some small trees in several areas (Photos 1 and 2).

Downstream Slope - The downstream slope is somewhat irregular and is overgrown with weeds, brush and saplings. In the area just below the gatehouse, there is some slight sloughing. A wet area exists at the right side of the slope at the berm. This is probably caused by runoff being caught in tire tracks on the berm. A seep with a flow of 15± gpm was noted at the toe of the embankment near the low-level outlet (See Sheet B-1). There is some erosion of the slope in the area of the seep (Photo 10). Water emanating from this seep was clear at the time of the inspection. Erosion of the left end of the slope is occurring due to a foot path up the slope in this area (Photo 4). Erosion from runoff is also occurring along the service road at the left abutment (Photo 4).

Spillway - The concrete weir has a large crack in the right end with horizontal movement of about 3 inches (Photo 5). There is also severe spalling and deterioration of the concrete with several pieces having cracked and fallen off. The concrete training wall at the left side of the spillway is damaged with some areas having exposed aggregate and reinforcement bars (Photo 7, Photo 8, Photo 9). The discharge channel has several areas where the lining is eroded, the largest of which appears in Photo 6. The bedrock forming the right spillway wall is visible in Photo 6 and 7.

c. Appurtenant Structures - The gatehouse features a concrete foundation and brick superstructure. The brick appears to be in good condition with no visible cracks. The gate house is presently boarded up for protection from recent vandalism and could not be inspected inside. The concrete foundation has severe erosion of the concrete (with exposed reinforcing bars) visible at the waterline. Erosion in this area measured approximately 1.5 feet in width, 6 inches deep and extended completely around the foundation.

The outlet structure is a concrete headwall at the downstream end of the left spillway training wall (See Sheet B-1, Photo 9).

d. Reservoir Area - The area surrounding the reservoir is steep-sided, wooded and undeveloped.

e. Downstream Channel - The downstream channel runs in the natural bed of Hop Brook. It flows through undeveloped wooded area to Route 309 and the initial impact area.

3.2 EVALUATION

Based upon the visual inspection, this dam is assessed as being in poor condition. The following features which could influence the future condition and/or stability of the dam were identified.

1. Erosion, spalling and cracks in the concrete at the gatehouse, spillway training wall, and downstream face of the weir will lead to further instability of these structures.

2. Seepage at the toe of the embankment could result in dam instability if material is carried from the embankment.

3. The growth of brush and small trees could, if unchecked, result in root penetration and weakening of the dam by uprooting or providing seepage paths through the embankment.

4. The gatehouse entrance should be more accessible for emergency operation of valves.

5. Continued erosion of the cover material along the foot paths at the top of the dam, upstream slope, downstream slope, and along the service road at the left end of the dam could lead to instability should the dam be overtopped.

SECTION 4: OPERATIONAL PROCEDURES

4.1 REGULATING PROCEDURES

a. General - No formal operation procedure exists although water company personnel visit the site daily to inspect the dam and grounds. The high-level inlet valve is left open and maintains flow through the dam. The mid-level inlets are opened when, on occasion it is desirable to significantly lower the water level.

b. Description of any Warning System in Effect - No formal warning system is in effect.

4.2 MAINTENANCE PROCEDURES

a. General - There is no formal maintenance program. However, brush is cut and debris removed from spillway on an as needed basis, and the dam is patrolled daily during the week.

b. Operating Facilities - No formal program for maintenance of operating facilities is in effect. However, the owner reports that greasing of the valves is performed as needed.

4.3 EVALUATION

A formal program of operation and maintenance procedures should be implemented, including documentation of lake levels for future reference. Also, a formal warning system should be developed within the time frame indicated in Section 7.1(c). Remedial operation and maintenance recommendations are presented in Section 7.

SECTION 5: EVALUATION OF HYDRAULIC/HYDROLOGIC

5.1 GENERAL

The watershed is 0.73 sq. miles of rolling and wooded terrain, of which a significant portion is undeveloped. The Tuller Reservoir Dam is basically a low spillage - high storage type project located on Hop Brook in the Connecticut River Basin. The maximum impoundment to the top of dam (El. 103.8) is estimated to be 175 Ac. Ft. The dam is classified as Intermediate in size and has a hazard classification of Significant.

The elevations used for the computations in Appendix D are N.G.V.D. so as to facilitate downstream flood routing computations. All elevations in this section have been converted to the assumed datum elevation to maintain unity in the text of this report. The assumed datum is based on the spillway crest = 100.0.

5.2 DESIGN DATA

No hydraulic/hydrologic design data or computations are available for this dam.

5.3 EXPERIENCE DATA

No information on any serious problem situations arising at the dam or downstream reaches of the dam was found although there was some repair to the spillway after the 1955 flood. The maximum previous discharge at this dam is unknown and it is reported that the dam had not been overtopped during the 1955 flood.

5.4 TEST FLOOD ANALYSIS

Based upon the U.S. Army Corps of Engineers "Preliminary Guidance For Estimating Maximum Probable Discharges" dated March, 1978; the watershed classification (rolling) and the drainage area of 0.73 square miles, a PMF of 1950 cfs or 2400 cfs per square mile is estimated at the damsite. The dam is classified as a significant hazard, intermediate size dam. Therefore, the test flood is in the ½ Probable Maximum Flood (½ PMF) to the full PMF range. Based on the degree of economic loss and potential for loss of life, the test flood for Tuller Reservoir Dam is selected as eqivalent to the PMF. The peak inflow is estimated at 1750 cfs and the peak outflow is 1630 cfs with the maximum stage in the reservoir at elevation 105.2, or 1.4 feet over the top of the dam. The spillway capacity is estimated to be 415 cfs with the pool to top of dam, which is 25% of the routed test flood outflow.

5.5 DAM FAILURE ANALYSIS

Utilizing the Corps of Engineers April 1978 "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow due to a breach of the dam is estimated to be 25,200 cfs with an estimated flood depth of 20 ft. immediately downstream of the dam. The prefailure flow in the stream is estimated to be 475 cfs.

The estimated peak flow rates and peak flood depth at three critical sections downstream of the dam resulting from a dam failure are:

Section (Ft. D/S of Dam)	Flow (CFS)	Flood Depth (Ft.)	Velocity (FPS)
At Dam	26,200	20	-
2500	13,900	10	8
3300	11,250	13	13
4250	8,200	8.5	9

A flood of this magnitude could wash out the culvert on Rt. 309 and flood one house located 11.5 feet above Hop Brook (south of Rt. 309 and opposite of Hedgehog Lane) with 1.5± Ft. of water. The water in the stream in this area is expected to rise from a depth of 3.4 feet just before the breach to a depth of 13.0 feet just after the breach, a raise in flood stage of 8.7 feet. In addition, two culverts downstream are expected to be impacted by this flood. However, if the culvert on Rt. 309 remains intact, then flood waters will follow Rt. 309, thereby impacting at least two houses, located at the corner of Hedgehog Lane and Rt. 309 with 1± ft. of water. The culvert on West Mountain Road would also be impacted (Sheet D-1, Appendix D-19 & 20).

Based on the hydraulic/hydrologic analysis and the potential for loss of less than a few lives at failure, the dam has a significant hazard classification.

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The dam is an earth embankment with a slope inclination of 2 horizontal to 1 vertical upstream and a berm along the center of the downstream slope. Above the berm the slope is 2 horizontal to 1 vertical and below the berm it is 3 horizontal to 1 vertical. The existance of a corewall is unknown, although the top of a 2 foot wide concrete wall is visable on the top of the dam from the left spillway training wall to the gatehouse. The upstream slope has riprap protection to 2.0 feet below the top of the dam and has a grass cover on the remainder of the embankment. No evidence of toe drains, piezometers or other seepage control and monitoring devices were found at the dam.

The visual inspection revealed a depression area on the top of the dam with some slight sloughing of the slope just below this area. A seep of 15± gpm exists at the toe of the embankment (seepage is clear) near the low-level outlet. There are also areas of erosion along the top of dam and deterioration of the concrete structures as described in Section 3. Recommendations are made in Section 7 for the above mentioned items and other items described in Section 3.

6.2 DESIGN AND CONSTRUCTION DATA

There is not enough design and construction data available to permit an in-depth assessment of the structural stability of the dam.

6.3 POST CONSTRUCTION CHANGES

The only post construction changes are the repairs to the spillway after the 1955 flood. No plans are available for these changes.

6.4 SEISMIC STABILITY

The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the dam is judged to be in poor condition. There are items which require repair, maintenance and monitoring. These include concrete repair, regrading slopes, removal of brush and seepage monitoring.

Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge" dated March, 1978 and hydraulic/hydrologic computations, peak inflow to the reservoir is 1750 cubic feet per second; peak outflow is 1630 cubic feet per second with the dam overtopped 1.4 feet. . The spillway capacity to the top of the dam (El. 103.8) is 415 cfs, which is equivalent to approximately 25% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based solely on visual inspection, history of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within 1 year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further investigation be made by a registered professional engineer qualified in dam design and inspection pertaining to the following items. Recommendations should be made by the engineer and implemented by the owner.

1. A detailed hydraulic/hydrologic analysis to determine the adequacy of the project discharge and the overtopping potential.

2. Inspection of gate valve facilities at the gatehouse and 18 inch low level outlet pipe. This examination should check to see that the valves can be seated properly, that there is no seepage into the outlet pipe from the embankment, and evaluate the condition of the gate house chamber.

3. Evaluation of the depression area on the top of the embankment and possible sloughing of the downstream slope in the area just below the gate house.

4. Origin and significance of seepage at the toe of the embankment near the low-level outlet. A monitoring program to evaluate variations of flow in this seep during varying reservoir levels should be developed and implemented. The erosion of the slope at this seep should be evaluated as well.

5. The origin and significance of the wet and soft area at the right portion of the berm on the downstream slope should be investigated.

6. Development of a program for rehabilitation of damaged concrete at the spillway weir, spillway training wall and gatehouse foundation.

7. Regrade the top of the dam and fill the area of erosion at the right end of the embankment near the spillway and visible concrete wall. Proper slope protection should be placed.

7.3 REMEDIAL MEASURES

a. Operation and Maintenance Procedures - The following measures should be undertaken within the time period indicated in Section 7.1c, and continued on a regular basis.

1. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference. A program for monthly inspection by the owner or owner representative should be developed and include proper documentation.

2. A comprehensive program of inspection by a registered professional engineer qualified in dam design and inspection should be instituted on a biennial basis.

3. The owner should develop and implement a formal emergency action plan and a downstream warning system, so in the event of emergencies at the dam, evacuation may be implemented in a prompt and organized manner.

4. Erosion on the downstream slope (foot path) and the service road at the left abutment should be regraded and proper slope protection place.

5. The seepage on the downstream slope should be monitored in accordance with the program recommended in Section 7.2.4.

6. Brush and small trees on the slopes should be removed.

7. The gatehouse should be restored to an accessible condition, but resistant to repeated vandalism.

7.4 ALTERNATIVES

One alternative to the above recommendations is to drain the reservoir and remove the dam.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT Tuller Reservoir Dam

DATE: May 9, 1980

TIME: 9:00 AM - 12:00 Noon

WEATHER: Sunny 70°F

W.S. ELEV. 100.1' U.S.

_____ U.S.

PARTY:	INITIALS:	DISCIPLINE:
1. Peter M. Heynen	PMH	Cahn - Geotech.
2. Milan Petrovsky	MP	Cahn - Geotech.
3. Murali Atluru	MA	DTC - Hydrology
4. Jay A Costello	JAC	Cahn - Geotech.
5. Jeffrey O. Borne	JB	Cahn - Geotech.
6. Tim Kavanaugh	TK	Cahn - Survey

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Embankment	PMH, MP, JAC, JB, TK	A-2
2.	Spillway	PMH, MP, MA, JAC, JB, TK	A-3
3.	Outlet Structure	PMH, JAC, JB, MA, MP, TK	A-4
4.	Gatehouse	PMH, JAC, JB	A-5
5.			
6.			
7.			
8.			
9.			
10.			
11.			
12.			

PERIODIC INSPECTION CHECK LIST

PROJECT Tuller Reservoir Dam DATE May 9, 1990

PROJECT FEATURE Embankment BY PMH, JAC, MP, JB, TK

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	103.8
Current Pool Elevation	100.2
Maximum Impoundment to Date	Unknown
Surface Cracks	None observed
Pavement Condition	
Movement or Settlement of Crest	
Lateral Movement	
Vertical Alignment	Appears good
Horizontal Alignment	
Condition at Abutment and at Concrete Structures	Some erosion of crest near spillway training wall
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Yes
Sloughing or Erosion of Slopes or Abutments	Erosion from footpaths, slight sloughing below gatehouse
Rock Slope Protection-Riprap Failures	None
Unusual Movement or Cracking at or Near Toes	None
Unusual Embankment or Downstream Seepage	Seepage @ 15± gpm on d/s slope
Piping or Boils	
Foundation Drainage Features	None observed
Toe Drains	
Instrumentation System	

PERIODIC INSPECTION CHECK LIST

PROJECT Tuller Reservoir Dam DATE May 9, 1980

PROJECT FEATURE Spillway BY PMH, MP, MA, JAC, JB, TK

AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	
Floor of Approach Channel	Sand, gravel, bedrock - good
b) Weir and Training Walls	
General Condition of Concrete	poor - cracking & deterioration of weir & training wall
Rust or Staining	None
Spalling	yes - weir and training wall
Any Visible Reinforcing	yes - training wall
Any Seepage or Efflorescence	None
Drain Holes	None
c) Discharge Channel	
General Condition	poor - deterioration training wall and floor concrete
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	
Floor of Channel	poor - holes in lining
Other Obstructions	None observed

PERIODIC INSPECTION CHECK LIST

PROJECT Tuller Reservoir Dam DATE May 9, 1980

PROJECT FEATURE Outlet BY PMH, JHC, JR, MR, MA, TK

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		poor
Rust or Staining		N/A
Spalling		yes - outlet pipe
Erosion or Cavitation		None observed
Visible Reinforcing		
Any Seepage or Efflorescence		
Condition at Joints		N/A
Drain Holes		
Channel		
Loose Rock or Trees Overhanging Channel		yes - trees
Condition of Discharge Channel		poor - debris, tires, boulders piled up in channel

PERIODIC INSPECTION CHECK LIST

PROJECT Tuller Reservoir Dam DATE May 9, 1980

PROJECT FEATURE Gatehouse BY PMH, JAC, JB

AREA EVALUATED	CONDITION
OUTLET WORKS-CONTROL TOWER	
a) **Concrete and Structural**	
General Condition	fair - foundation needs repair
Condition of Joints	good
Spalling	foundation concrete
Visible Reinforcing	
Rusting or Staining of Concrete	None observed
Any Seepage or Efflorescence	
Joint Alignment	good
Unusual Seepage or Leaks in Gate Chamber	None observed
Cracks	Some - foundation
Rusting or Corrosion of Steel	yes
b) **Mechanical and Electrical**	
Air Vents	
Float Wells	
Crane Hoist	
Elevator	Not observed
Hydraulic System	
Service Gates	
Emergency Gates	
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE

TULLER RESERVOIR DAM

EXISTING PLANS

None Available.

SUMMARY OF DATA AND CORRESPONDENCE

No information is available.

APPENDIX C

DETAIL PHOTOGRAPHS

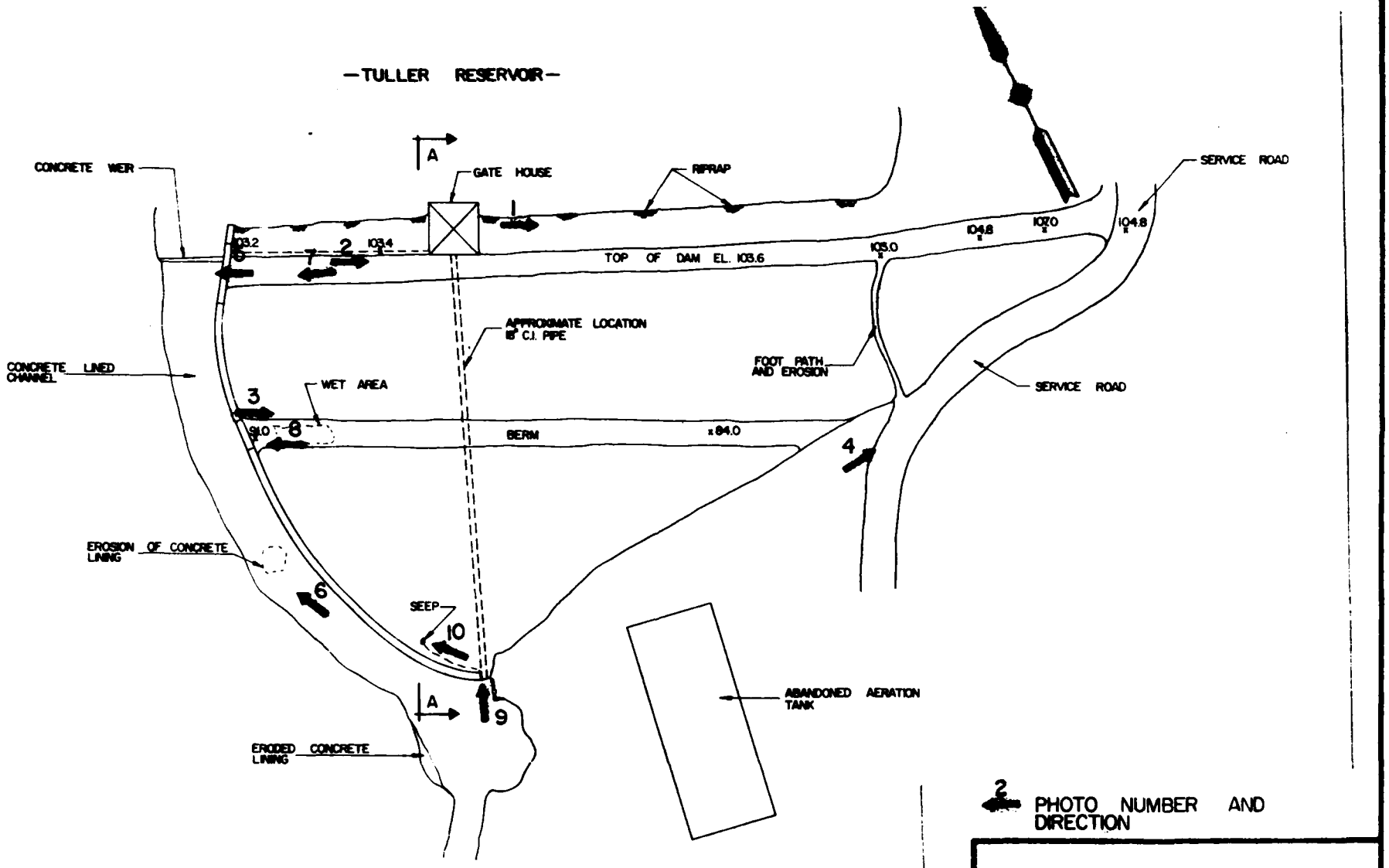

-TULLER RESERVOIR-
A
CONCRETE WEIR
GATE HOUSE
RIPRAP
SERVICE ROAD
103.2
103.4
TOP OF DAM EL. 103.6
105.0
104.8
107.0
104.8
APPROXIMATE LOCATION 18" C.I. PIPE
CONCRETE LINED CHANNEL
WET AREA
FOOT PATH AND EROSION
SERVICE ROAD
BERM
94.0
EROSION OF CONCRETE LINING
SEEP
ABANDONED AERATION TANK
A
ERODED CONCRETE LINING
PHOTO NUMBER AND DIRECTION
PHOTO LOCATION PLAN
TULLER RESERVOIR DAM
SHEET C-1

Photo 1 - Upstream slope and riprap, (May, 1980)

Photo 2 - Top of dam from left spillway training wall. Gate house at center of photo, (May, 1980).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Tuller Reservoir Dam
	INSPECTION OF	Hop Brook
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	NON-FED. DAMS	Simsbury, CT
		CE # 27 785 KE
		DATE Sept. 1980 PAGE C-1

Photo 3 - Downstream slope from right abutment. Berm is barely visable parallel to top of dam along center of photo, (May, 1980).

Photo 4 - Foot Path on left end of downstream slope and erosion at service road along left abutment, (May, 1980).

Photo 6 - Concrete lined spillway discharge channel along right abutment. Erosion of lining at center and ledge at left side of channel, (May, 1980).

Photo 5 - Concrete trapezoidal weir at right end of embankment. Note cracking and horizontal movement starting at concrete block and extending diagonal toward downstream, (May, 1980).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF	Tuller Reservoir Dam
	INSPECTION OF	Hop Brook
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	NON-FED. DAMS	Simsbury, CT
		CE# 27 785 KE
		DATE Sept. 1980 PAGE C-3

Photo 7- Left spillway training wall and right end of embankment. Note concrete deterioration of training wall and visable concrete wall extending along crest from training wall, (May, 1980).

Photo 8 - Deterioration of left training wall along spillway discharge channel. Ledge in background is at opposite side of channel, (May, 1980).

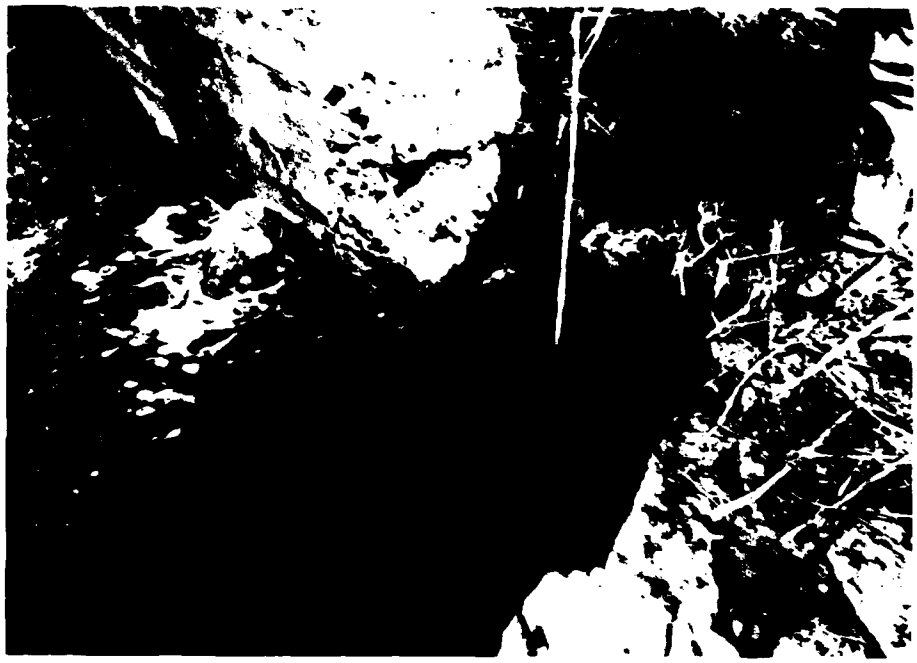


Photo 9 - Deterioration of left spillway training wall and low-level outlet at center of photo, (May, 1980).

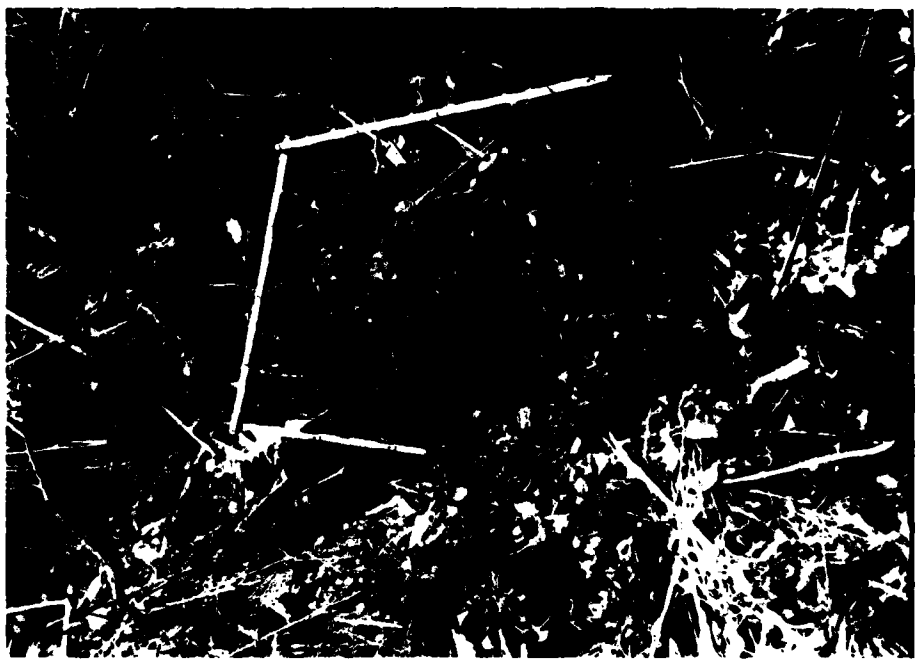
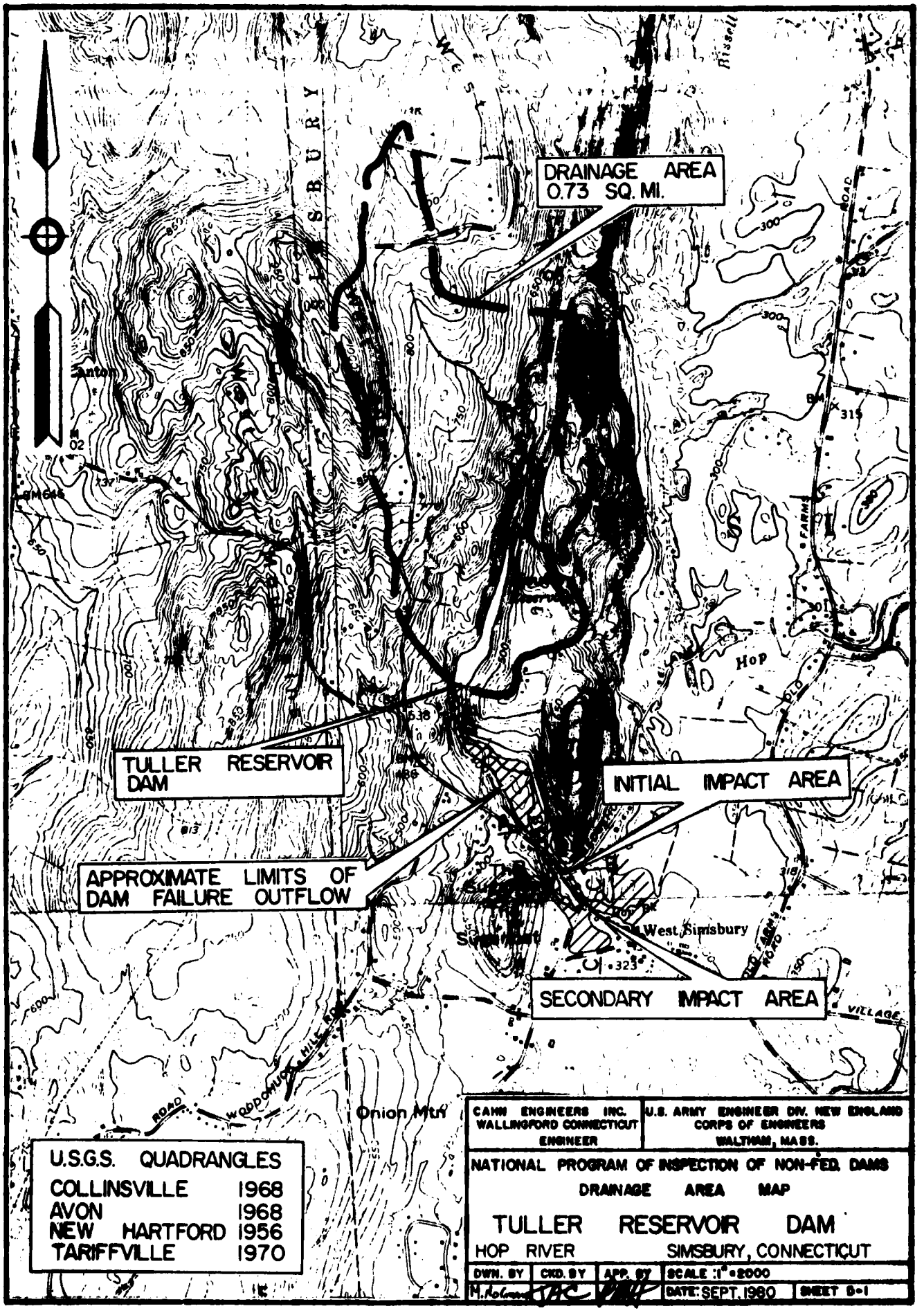


Photo 10 - Seepage on downstream slope at right end of dam. Flow rate is approximately 15 gpm, (May, 1980).


DRAINAGE AREA
0.73 SQ. MI.
TULLER RESERVOIR
DAM
INITIAL IMPACT AREA
APPROXIMATE LIMITS OF
DAM FAILURE OUTFLOW
SECONDARY IMPACT AREA
U.S.G.S. QUADRANGLES
COLLINSVILLE 1968
AVON 1968
NEW HARTFORD 1956
TARIFFVILLE 1970
CAHN ENGINEERS INC.
WALLINGFORD CONNECTICUT
ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
TULLER RESERVOIR DAM
HOP RIVER
SIMSBURY, CONNECTICUT
DWN. BY
CKD. BY
APP. BY
SCALE :1" = 2000
DATE: SEPT. 1980
SHEET 0-1

PROBABLE MAXIMUM FLOOD (PMF) DETERMINATION

DRAINAGE AREA — 0.73 Sq. MILES FROM CONN STATE DEP BULLETIN NO. 1, 1972, (GAZETTEER OF NATURAL DRAINAGE AREAS, P. 31).

WATERSHED CLASSIFICATION — "ROLLING" BASED UPON USGS MAP AND SITE VISITS.

PMF PEAK INFLOW —

BY EXTRAPOLATION OF THE CORPS OF ENGINEERS DECEMBER 1977 GUIDE CURVES, FOR ABOVE CONDITIONS PEAK FLOW RATE SELECTED = 2400 CFS/Sq. MILE

∴ PMF PEAK INFLOW = 2400 × 0.73 = 1750 CFS

SIZE CLASSIFICATION —

FOR THE PURPOSE OF DETERMINING PROJECT SIZE, THE MAXIMUM STORAGE ELEVATION IS CONSIDERED EQUAL TO THE TOP OF DAM.

TOP OF DAM	= EL.	453.8*
INVERT OF OUTLET PIPE	= EL.	408.3
HEIGHT OF DAM		45.5 FT.

* THE RESERVOIR ELEVATION IS NOT INDICATED ON THE USGS MAP. HOWEVER, EXAMINING THE CONTOURS ON THE USGS MAP, THE NORMAL RESERVOIR ELEVATION IS ASSUMED TO BE 450.0 NGVD AND IS ASSUMED TO BE THE SAME FOR THE SPILLWAY CREST. ALL OTHER ELEVATIONS ARE REFERENCED TO THIS ASSUMED ELEVATION AND ARE OBTAINED BASED ON INFORMATION GATHERED DURING SITE VISITS.

PLANIMETERING FROM USGS MAP FOR RESERVOIR SURFACE AREAS—

AT EL. 450 (SPILLWAY CREST) = 9.6 ACRES
AT EL. 460 = 14.2 ACRES
AT EL. 470 = 18.4 ACRES

A STAGE-RESERVOIR AREA CURVE IS PLOTTED (SHEET 3)
FROM THIS CURVE, RESERVOIR AREA AT TOP OF DAM = 11.2 ACRES
AVERAGE RESERVOIR AREA BETWEEN SPILLWAY CREST AND TOP OF DAM = 10.4 AC.
∴ STORAGE BETWEEN SPILLWAY CREST AND TOP OF DAM = 3.8 × 10.4 = 39.5 AC.FT.

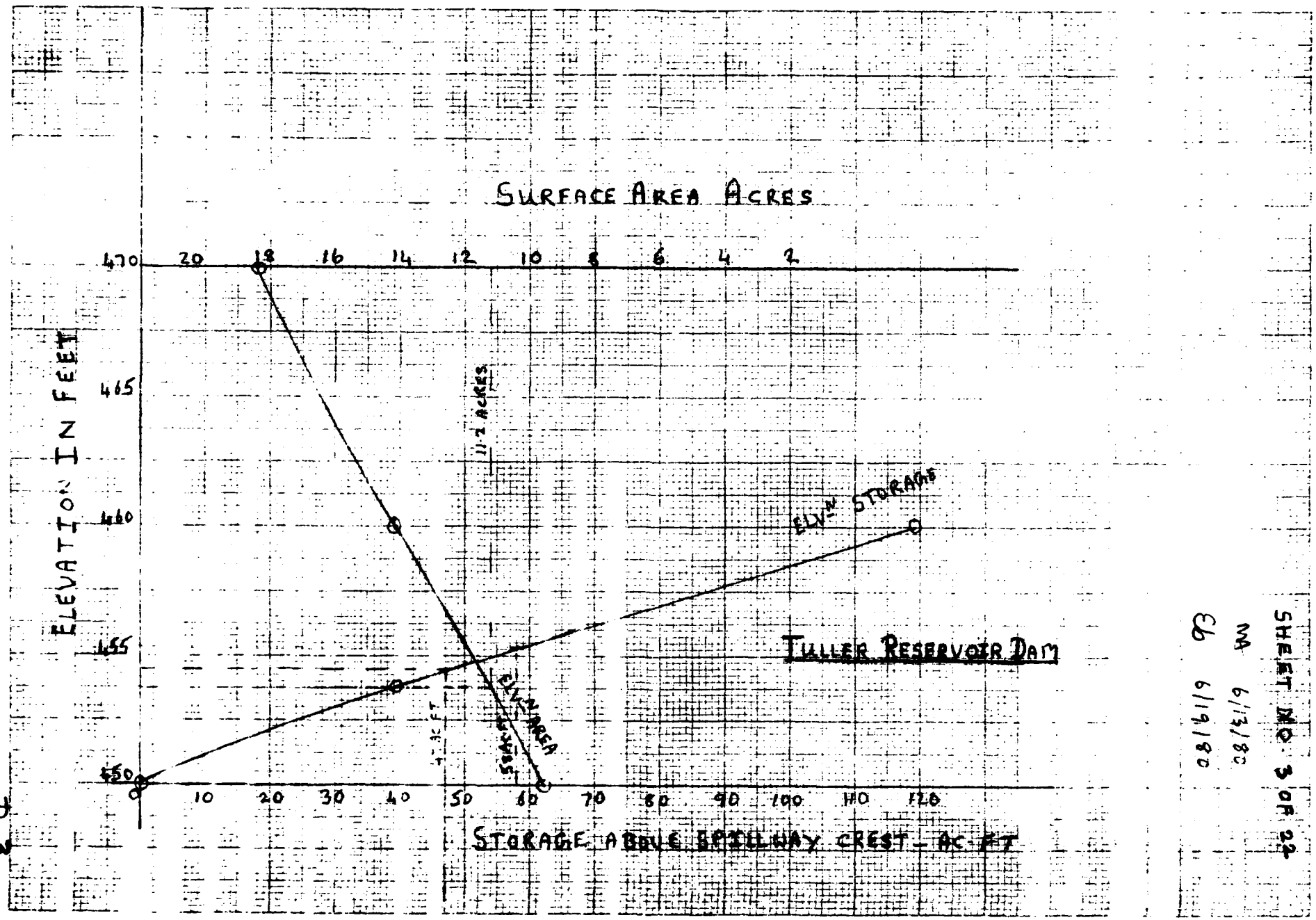
ESTIMATED STORAGE BELOW SPILLWAY CREST
$= \frac{1}{3} \times 9.6 \times 41.7 = 133.5$ AC.FT.

(EL. 450.0 − EL. 408.3 = 41.7')

∴ MAXIMUM IMPOUNDMENT TO TOP OF DAM = 39.5 + 133.5 = 173 AC.FT.

A STAGE-STORAGE CURVE IS PLOTTED ON SHEET 3.

THUS, ACCORDING TO CORPS OF ENGINEERS GUIDELINES TABLE 1, THE TULLER RESERVOIR DAM IS CLASSIFIED INTERMEDIATE BASED UPON THE HEIGHT OF DAM WHICH IS 45.5' (> 40' AND < 100') EVEN THOUGH THE STORAGE CAPACITY IS ONLY 173 AC.FT (< 1000 AND > 50).



D-3

SHEET NO. 3 OF 22

6/13/80

6/19/80

HAZARD POTENTIAL—
SIGNIFICANT BASED ON DAM BREACH ANALYSIS AND RELATIVE LOCATIONS OF HOUSES AND OTHER STRUCTURES. A DETAILED DISCUSSION OF HAZARD POTENTIAL IS INCLUDED AT THE END OF BREACH ANALYSIS SECTION OF APPENDIX D.

SELECTION OF TEST FLOOD—

FOR THE INTERMEDIATE SIZE AND SIGNIFICANT HAZARD POTENTIAL CLASSIFICATION, TABLE 3 OF CORPS OF ENGINEERS RECOMMENDED GUIDELINES, THE TEST FLOOD COULD BE IN THE $\frac{1}{2}$ PMF TO PMF RANGE.
BASED ON THE INVOLVED RISK POTENTIAL (HOUSES & STATE HIGHWAY) A TEST FLOOD = PMF IS SELECTED.
TEST FLOOD PEAK INFLOW = 1750 CFS.

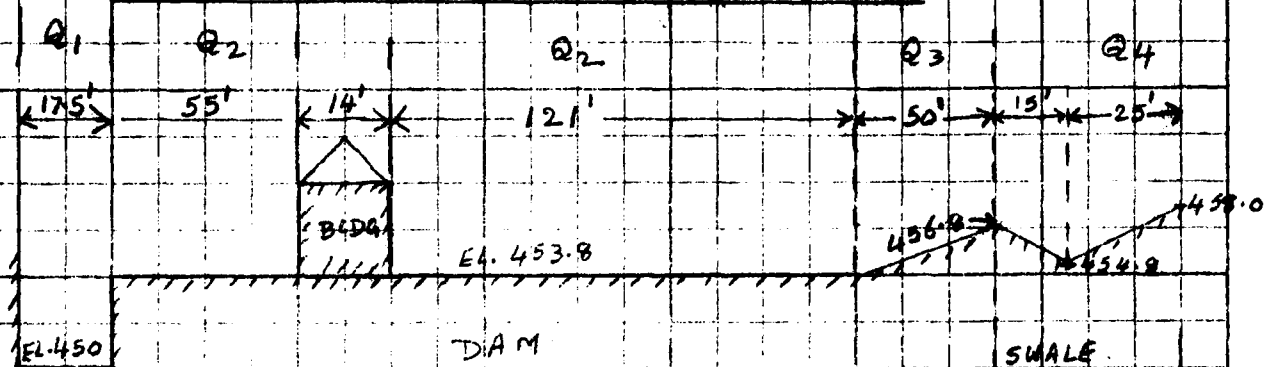
PMF WOULD RESULT FROM 19" RUN-OFF FROM 0.73 SQ. MILES OF DRAINAGE AREA.

$\therefore$ TOTAL STORM VOLUME $= \frac{19}{12} \times 0.73 \times 640 = 740$ AC FT

THUS, MAXIMUM STORAGE (BETWEEN SPILLWAY CREST AND TOP OF DAM) OF 39.5 AC.FT IS ONLY 5% OF THIS STORM VOLUME.

NOTE: SURCHARGE STORAGE ROUTING IS PERFORMED FOR A PEAK FLOOD OF $\frac{1}{2}$ PMF ALSO

COMPOSITE DISCHARGE RATING CURVE



POTENTIAL OVERFLOW SECTION

$Q_1 = CLH^{3/2}$
$= 56H^{3/2}$

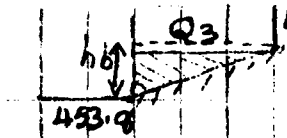
CR. EL. 450.0, L = 17.5'
C = 3.2 (REF: USGS BOOK 3, CHAPTER A5, P16, FIGURE 7)

DAM $Q_2 = CLH^{3/2}$
$= 493H^{3/2}$

L = 55' + 12' = 176' (14' LENGTH OF BUILDING IS NOT EXPECTED TO CONTRIBUTE TO FLOW)
C = 2.8 ASSUMED (GRASS & SMALL BRUSH)

INCLINED SECTION OF DAM

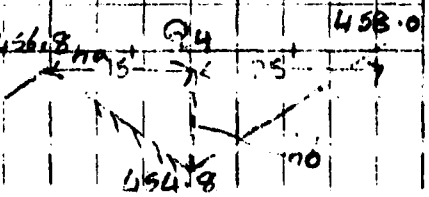
$$Q_3 = \frac{2}{5} C \frac{L}{(h_b - h_a)} \left(h_b^{5/2} - h_a^{5/2}\right)^{*}$$



UP TO EL. 456.8, ha = 0
ASSUME C = 2.7, L = 50

SWALE

SIMILARLY Q_4 IS CALCULATED BY USG METHOD - ha = 0 UP TO EL. 456.8, & EL. 459.0 AT THE TWO ENDS; C = 3.7 ASSUMED.



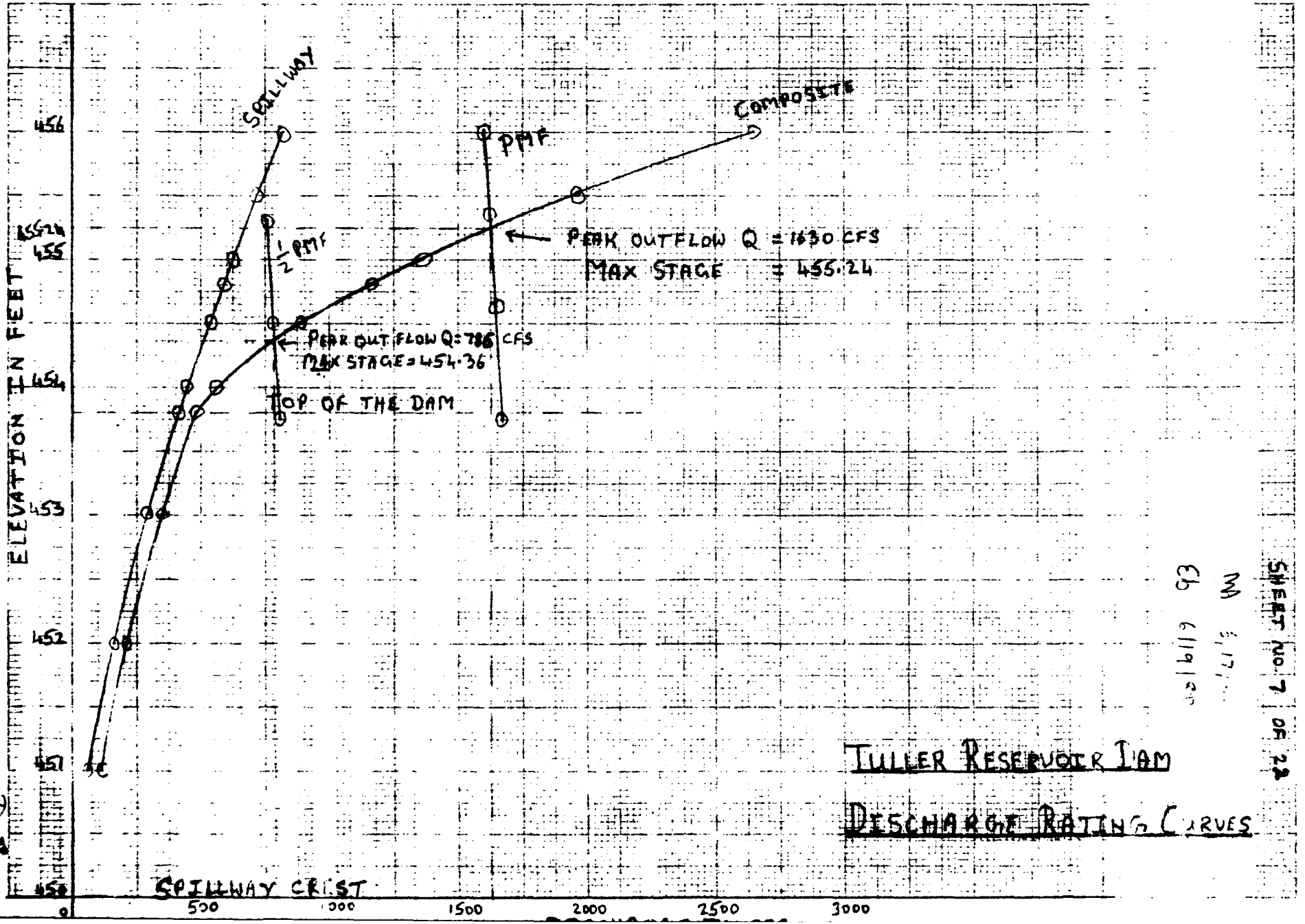
* NOTE USGS RECOMMENDED FORMULA FOR MORE PRECISE DISCHARGE OVER INCLINED DAM/EMBANKMENT CREST. (REF: MEASUREMENT OF PEAK DISCHARGES AT DAMS BY INDIRECT METHODS, USGS BOOK 3, CHAPTER A5, PAGES 3-4, 1968)

OUTLET PIPE

18" OUTLET PIPE WITH INVERT AT EL. 408.3 USING $Q_5 = CA\sqrt{2gH}$ AND APPROXIMATELY ACCOUNTING FOR LOSSES IN PIPE, Q_5 VALUES ARE COMPUTED AND TABULATED FOR HEAD ABOVE THE CENTER OF PIPE (EL. 409.05)

ELVN	Q_1 SPILLWAY EL. 450	Q_2 - DAM EL. 453.8	Q_3 - INCLINED SECTION OF DAM EL. 453.8	Q_4 SWALE EL. 454.8	Q_5 OUTLET EL 409.05	TOTAL Q CFS
451	56	0	0	0	59	115
452	158	0	0	0	60	218
453	291	0	0	0	60	351
TOP OF DAM 453.8	415	0	0	0	61	476
454	448	44	1	0	61	554
454.5	535	289	7	0	61	892
454.8	589	493	18	0	62	1162
455	626	648	28	1	62	1365
TEST FLOOD 455.24	670	850	46	2	62	1630
455.5	722	1093	68	7	62	1952
456	823	1609	129	26	62	2649
½ PMF 454.36	510	209	5	0	61	785

DISCHARGE RATING CURVES FOR OUTLET CONDUIT AND SPILLWAY ARE PLOTTED ON [illegible]



TULLER RESERVOIR DAM

DISCHARGE RATING CURVES

SHEET NO. 7 OF 22

6/19/03 MW

DETERMINATION OF PEAK OUTFLOW

SHORT CUT ROUTING OF RESERVOIR — FOR TEST FLOOD INFLOW
CORPS OF ENGINEERS GUIDELINES "SURCHARGE STORAGE ROUTING"
ALTERNATE METHOD USED.
FOR 1750 CFS (PMF) THE DISCHARGE RATING CURVE
GIVES ELVN = 455.35 AND
FROM STAGE-STORAGE CURVE FOR THIS ELVN
STORAGE = 58 AC.FT.

$$STOR_i = \frac{58 \times 12}{0.73 \times 640} = 1.49''$$ OF RUN OFF

$$Q_{P_i} = Q_P \left(1 - \frac{STOR_i}{19}\right)$$

① STOR$_i$ INCHES	② $\left(1 - \frac{STOR_i}{19}\right)$	③ STOR$_i$ AC FT ① $\times \frac{0.73 \times 640}{12}$	④ Q_{P_i} CFS ② $\times$ 1750	⑤ ELVN FROM STORAGE CURVE USING ③
1.00	0.95	39	1670	453.75
1.25	0.74	49	1650	454.62
1.49	0.93	58	1625	455.35
1.75	0.91	68	1600	456

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE
PEAK OUTFLOW Q = 1630 CFS
MAXIMUM STAGE = 455.24

TOP OF DAM = EL. 453.8

∴ THE DAM IS OVERTOPPED BY 1.44 FEET

ROUTING IS ALSO DONE FOR ½ PMF

½ PMF FLOW = 1750 × ½ = 875, SAY 900 CFS

DETERMINATION OF PEAK OUTFLOW –

SHORTCUT ROUTING OF RESERVOIR –

FOR 900 CFS (½ PMF) INFLOW THE DISCHARGE RATING CURVE GIVES ELVN = 454.5
FROM STAGE-STORAGE CURVE FOR THIS ELVN STORAGE = 47 AC.FT.

$STOR_i = \frac{47 \times 12}{0.73 \times 640} = 1.2''$ OF RUN-OFF

$Q_{p_i} = Q_{p_1}\left(1 - \frac{STOR_i}{9.5}\right)$

① $STOR_i$ INCHES	② $\left(1 - \frac{STOR_i}{9.5}\right)$	③ $STOR_i$ AC.FT. ① × 0.73 × 640 / 12	④ Q_{p_i} CFS ③ × 900	⑤ ELVN FROM STORAGE CURVE USING ③
1.00	0.89	39	801	453.75
1.2	0.87	47	783	454.5
1.5	0.84	58	756	455.3

COLUMNS ④ & ⑤ ARE PLOTTED ON DISCHARGE RATING CURVE

PEAK OUTFLOW Q = 785 CFS
MAX. STAGE = 454.36
TOP OF DAM = EL. 453.8

∴ THE DAM IS OVERTOPPED BY 0.56 FEET.

BREACH ANALYSIS - DOWNSTREAM FAILURE HAZARD

BREACH OUTFLOW $Q_b = \frac{8}{27} \times W_b \times \sqrt{g} \times y_o^{3/2}$

y_o = 45.5'

W_b = 40% OF MID-HEIGHT LENGTH OF THE DAM.
= 40% OF 125' = 50' (MID-HEIGHT L FROM CE FIELD INFORMATION)

$Q_b = \frac{8}{27} \times 50 \times \sqrt{32.2} \times (45.5)^{3/2}$ = 25,780 CFS

PEAK FAILURE OUTFLOW (Q_{P_1}) = Q_b + SPILLWAY DISCHARGE WITH POOL AT TOP OF DAM.
= 25,780 + 415* ≅ 26,200 CFS.

ESTIMATED FAILURE FLOOD DEPTH IMMEDIATELY D/S FROM DAM ≅ 0.44 y_o = 0.44 × 45.5 = 20 Ft.

PERFORM DOWNSTREAM ROUTING OF PEAK FAILURE OUTFLOW

SELECT A SECTION AA 2500' DOWN STREAM OF THE DAM JUST ABOVE THE CULVERT ON ROUTE 309.

USING MANNING'S EQUATION

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$

$72.75 \times A \times R^{2/3}$

WHERE n = 0.07 ASSUMED FOR THE CHANNEL WHICH IS WINDING AND HAS COBBLES AND BOULDERS.
FRICTION SLOPE = 0.017 ESTIMATED FROM USGS MAP

*THE DISCHARGE FROM THE 18" OUTLET PIPE IS NOT INCLUDED, SINCE THE OUTLET PIPE IS EXPECTED TO BE WITHIN THE BREACH WIDTH.

ELVN	A SQ FT	P	R = A/P	$R^{2/3}$	Q CFS
360	0	—	—	—	—
365	462	185	2.5	1.84	2,340
370	1825	365	5	2.93	14,700
372	2577	387	6.7	3.54	25,100
375	3789	421	9	4.33	45,100

STAGE-AREA AND STAGE-DISCHARGE CURVES ARE PLOTTED FOR SECTION AA FOR PEAK FAILURE OUTFLOW Q_{P_1} = 26,200 CFS

ELVN = 372.3 FROM STAGE-DISCHARGE CURVE

AND STAGE AREA CURVE GIVES AREA ≅ 2700 SQ.FT

VOLUME OF REACH $V_1 = \frac{1750^{*} \times 2700}{43,560} = 108.5$ AC.FT.

TRIAL $Q_{P_2} = Q_{P_1}\left(1 - \frac{V_1}{S}\right)$, WHERE S = TOTAL STORAGE TO TOP OF DAM

$= 26,200\left(1 - \frac{108.5}{173}\right) \cong 9800$ CFS

FOR THIS Q_{P_2} THE STAGE-DISCHARGE CURVE GIVES

ELVN = 368.5 AND AREA ≅ 1350 SQ.FT.

$\therefore V_2 = \frac{1750 \times 1350}{43,560} = 54$ AC.FT.

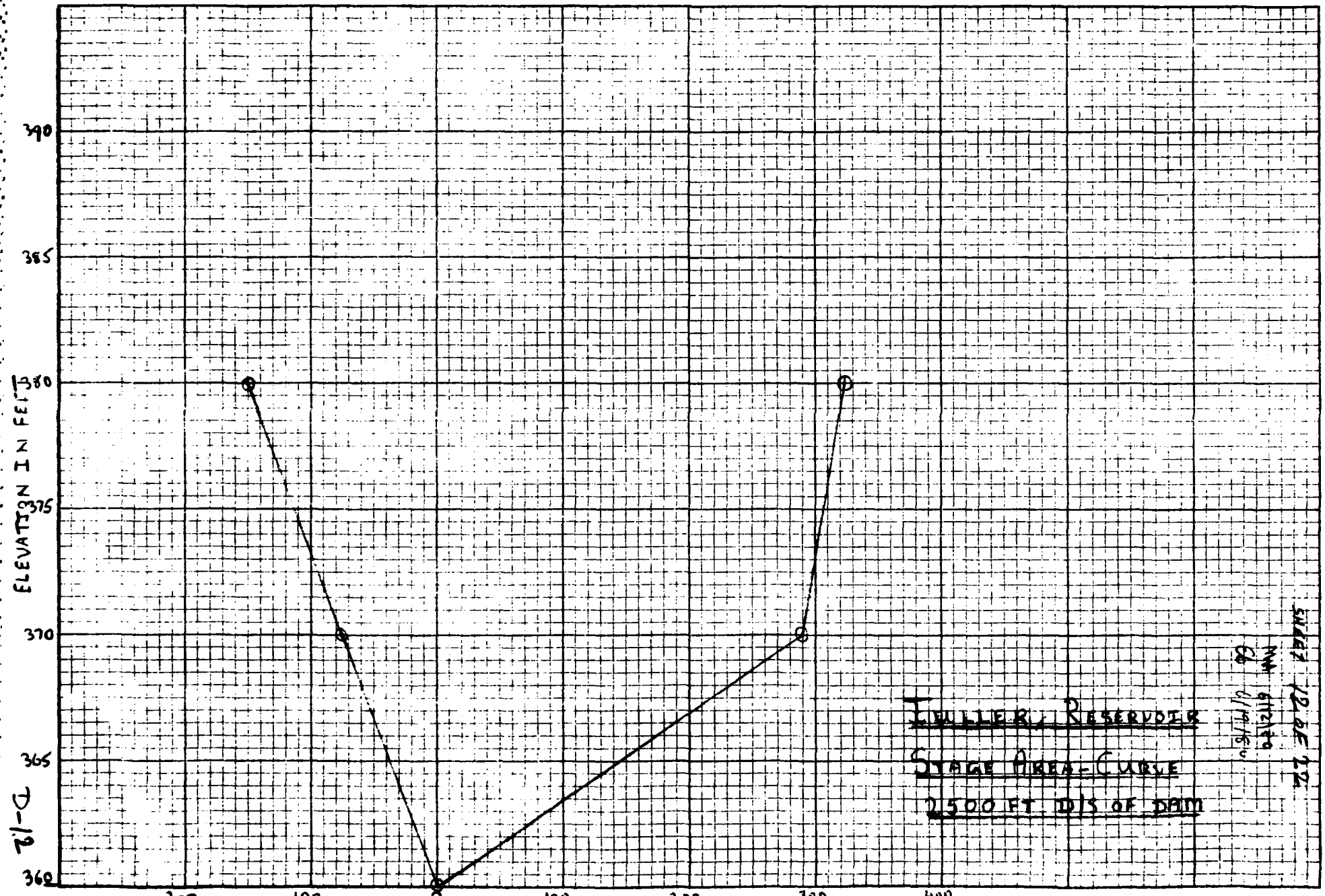
RECOMPUTING $Q_{P_2} = 26,200\left(1 - \frac{108.5+54}{173}\right) \cong 13,900$ CFS

FLOOD DEPTH AT SECTION AA = EL. 370 − EL. 360 = 10 FT

FLOOD STAGE AT SECTION AA ≅ 370

AND VELOCITY AT SECTION AA = $\frac{13,900}{1800} \cong 8$ FPS

* SINCE THE FIRST 750 FT. OF THIS CHANNEL REACH IS NARROW AND HAS STEEP SLOPE, IT IS ASSUMED THAT NO STORAGE VOLUME WOULD BE ATTENUATED IN THIS INITIAL SECTION.

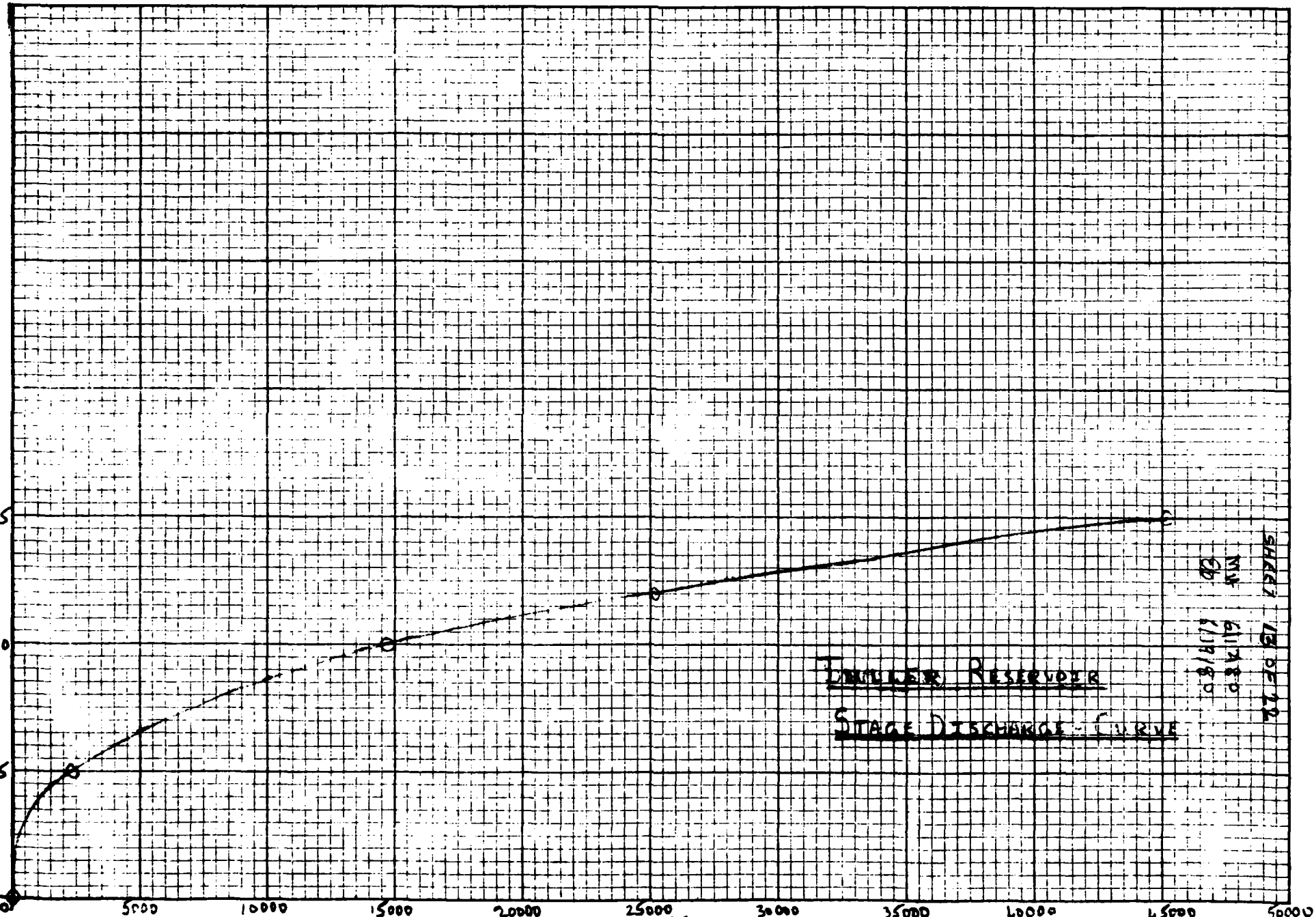


FULLER RESERVOIR
STAGE AREA - CURVE
2500 FT. D/S OF DAM

SECTION - AA

SHEET 12 OF 22
MM 6/12/69
6/19/69

D-12



TRAILER RESERVOIR

STAGE DISCHARGE CURVE

SECTION—AA

SHEET 13 OF 22

MAY 6/1/80

D-13

SELECTING A SECTION BB 800' D/S OF AA

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$ $n = 0.07$ ASSUMED

$= 4.03 \times A \times R^{2/3}$ $S = 0.037$ ESTIMATED FROM USGS MAP

EL	A SQ.FT	P	R = A/P	$R^{2/3}$	Q CFS
330	0	—	—	—	—
335	125	51	2.45	1.82	920
340	500	102	4.9	2.89	5800
345	1250	202	6.19	3.37	17,000

STAGE-AREA AND STAGE DISCHARGE CURVES ARE PLOTTED. FOR $QP_1 = 13,900$ CFS, EL^n 344 AND FROM STAGE AREA CURVE, AREA = 1060 SQ.FT

VOLUME OF REACH $V_1 = \frac{800 \times 1060}{43,560} = 19.5$ AC.FT

STORAGE REMAINING $= 173 - \frac{108.5 + 64}{2} \cong 92$ AC.FT

TRIAL $QP_2 = QP_1 \left(1 - \frac{V_1}{S}\right)$

$= 13,900 \left(1 - \frac{19.5}{92}\right) = 10,950$ CFS

FOR 10,950 CFS, ELV^n = 342.7 AND AREA = 843 SQ.FT.

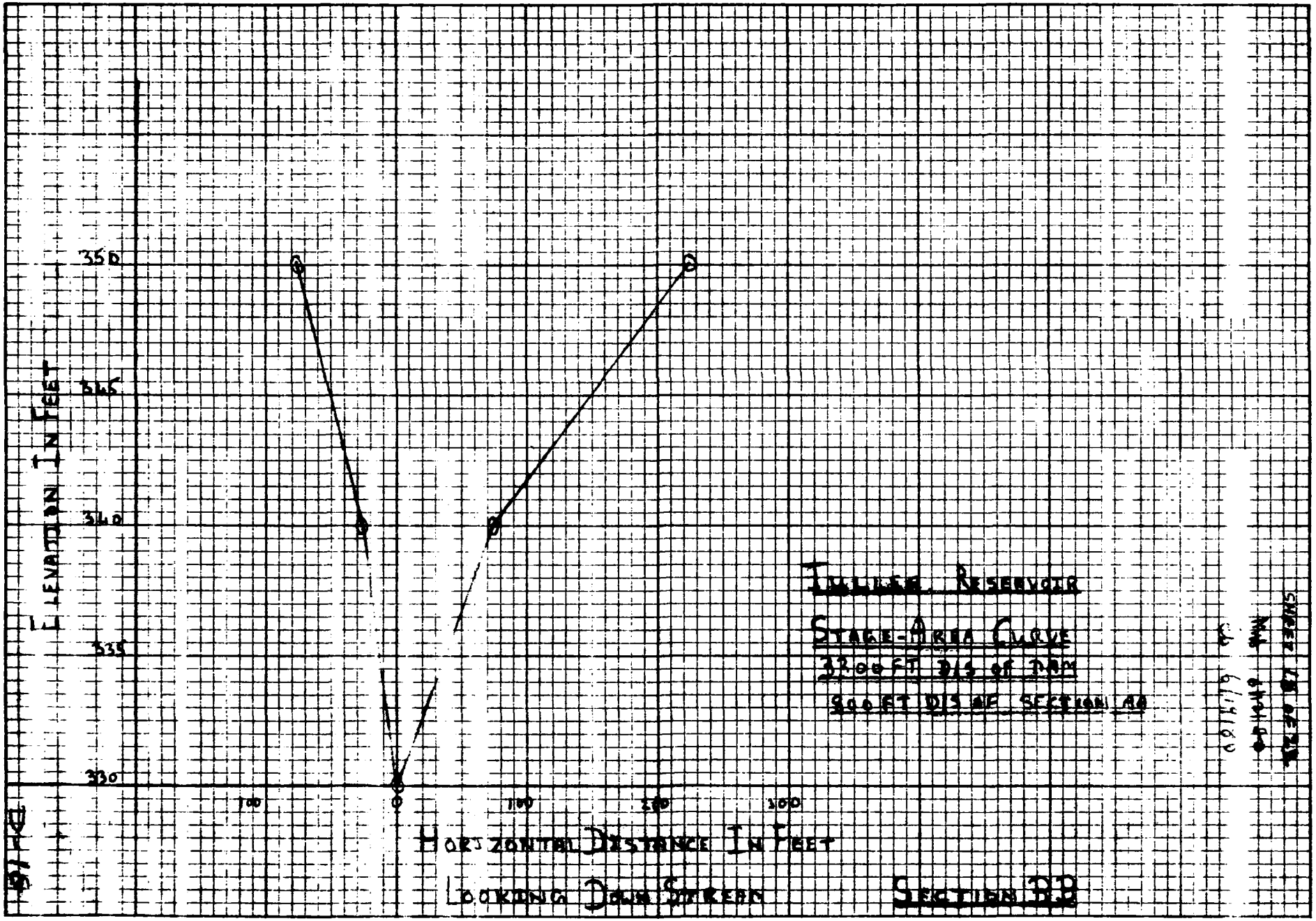
$V_2 = \frac{800 \times 843}{43,560} = 15.5$ AC.FT

RECOMPUTING $QP_2 = 13,900 \left(1 - \frac{\frac{19.5+15.5}{2}}{92}\right) = 11,250$ CFS

FLOOD STAGE $\cong 343$

DEPTH OF FLOOD WATER = EL 343 − EL 330 = 13 FT AT SECTION BB

VELOCITY AT SECTION BB = $\frac{11,250}{875} \cong 13$ FPS



IMLILAKE RESERVOIR
STAGE-AREA CURVE
3200 FT. D/S OF DAM
900 FT DIS OF SECTION AA

SHEET 14 OF 24 SHEETS

SELECT A SECTION CC 950' DIS OF BB

$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$

$= 4.16 \times A \times R^{2/3}$

$n = 0.05$ ASSUMED FOR THIS REACH OF CHANNEL WHICH IS FAIRLY UNIFORM WITH SOME GRASS

$S = 0.021$ ESTIMATED FROM USGS MAP

ELV^N	A - SQ.FT	P	R = A/P	$R^{2/3}$	Q_1, CFS
310	0	—	—	—	—
315	312	125	2.5	1.84	2,390
320	1250	250	5	2.93	15,200

STAGE, AREA AND STAGE-DISCHARGE CURVES ARE PLOTTED. FOR Q_{P_1} = 11,250 CFS, ELVN = 319.2 AND FROM STAGE-AREA CURVE, AREA = 1048 SQ.FT.

VOLUME OF REACH $V_1 = \frac{950 \times 1048}{43,560} \cong 23$ AC.FT

STORAGE REMAINING = $92 - \frac{19.5 + 15.5}{2} \cong 75$ AC.FT.

TRIAL $Q_{P_2} = Q_{P_1}\left(1 - \frac{V_1}{S}\right) = 11,250\left(1 - \frac{23}{75}\right) = 7800$ CFS

FOR 7800 CFS, ELVN = 318.2 AND A = 820 SQ.FT.

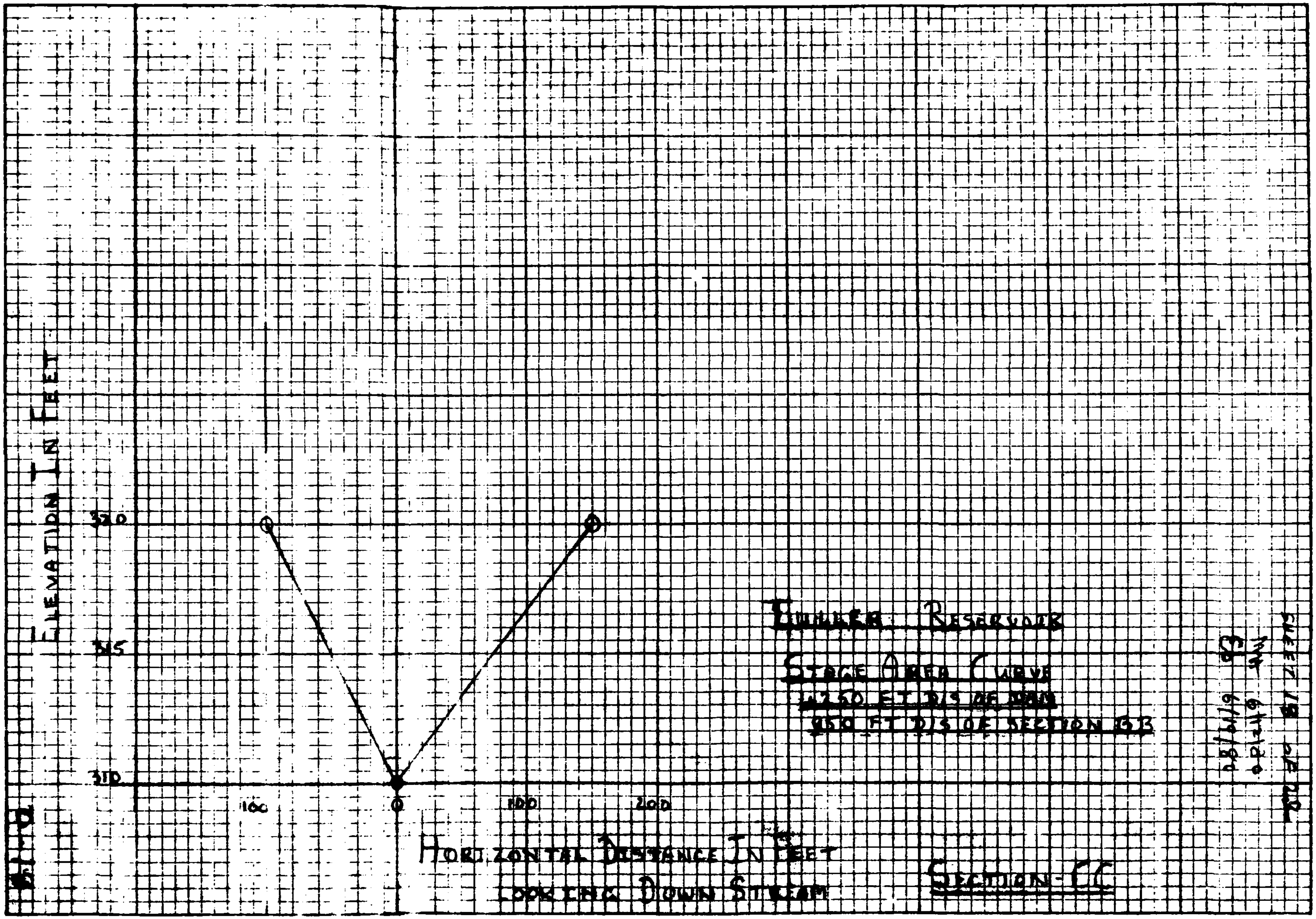
$V_2 = \frac{950 \times 820}{43,560} = 18$ AC.FT.

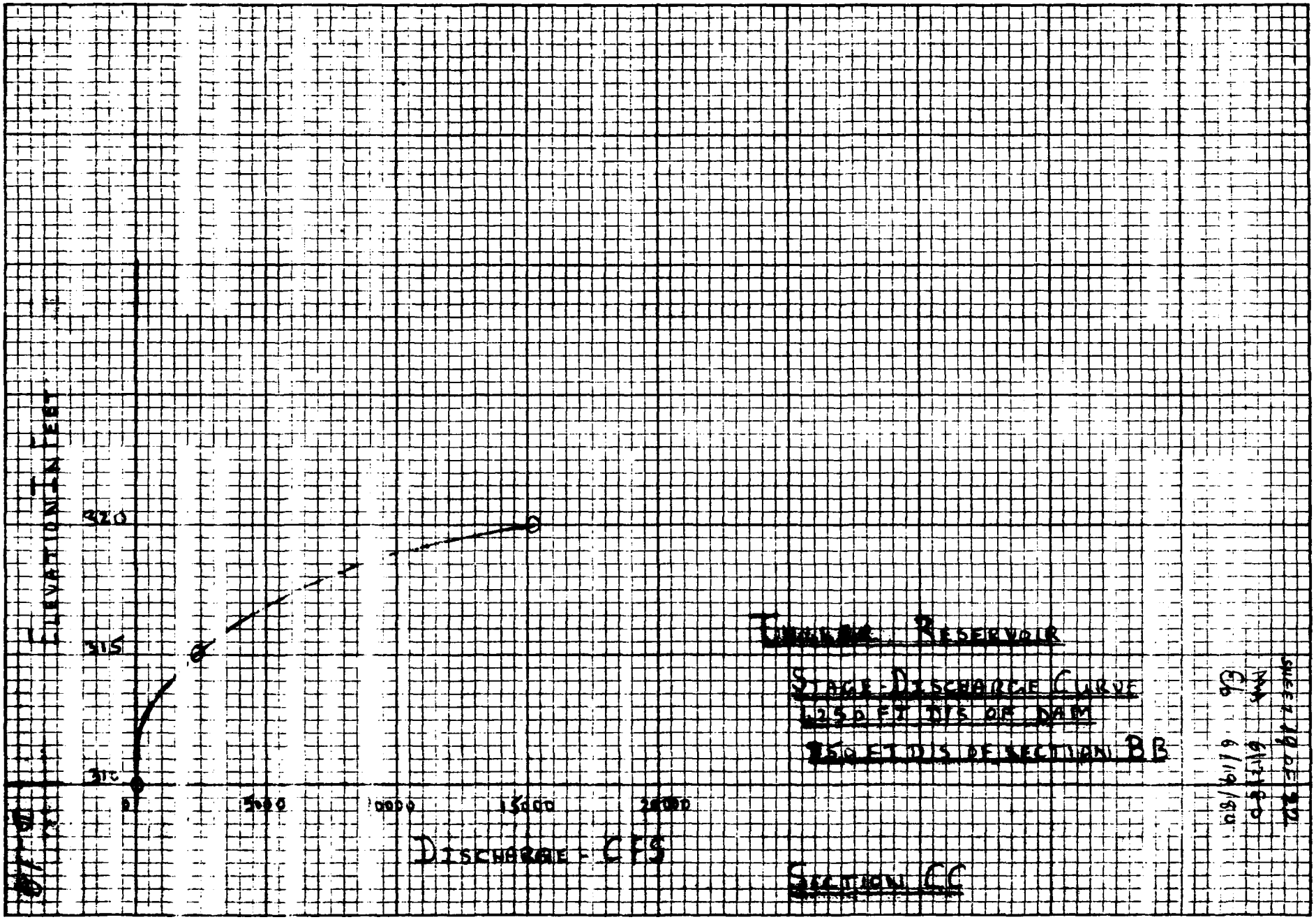
RECOMPUTING: $Q_{P_2} = 11,250\left(1 - \frac{\frac{23+18}{2}}{75}\right) = 8200$ CFS

AVG FLOOD STAGE $\cong 318.5$

∴ DEPTH OF FLOOD WATER = 318.5 − 314 = 4.5 ft. + C

VELOCITY IN SECTION CC = $\frac{8200}{890} \cong 9$ FPS





TURNER RESERVOIR
STAGE DISCHARGE CURVE
1250 FT D/S OF DAM
1500 FT D/S OF SECTION BB
SECTION CC

SHEET 19 OF 20
JWS 6/19/80
CD 6/19/80

FAILURE HAZARD POTENTIAL

SUMMARY OF BREACH ANALYSIS RESULTS—

LOCATION	DISTANCE FROM DAM FT.	PEAK FAILURE RATE CFS	CHANNEL BED EL	FLOOD STAGE	FLOOD DEPTH FT	VELOCITY FPS.
DAM	0	26,200	408.3	428.3	20	—
AA	2500	13,900	360	370.0	10	8
BB	3300	11,250	330	343.0	13	13
CC	4250	8,200	310	318.5	8.5	9

SECTION AA IS TAKEN JUST ABOVE THE 12'x6' CONCRETE CULVERT ON ROUTE 309 AND AT DAM BREACH ONE OF THE TWO SITUATIONS COULD ARISE—

a) THE CULVERT COULD BE WASHED OUT— WHEREUPON, THE FLOOD DEPTH IN THE CHANNEL ADJACENT TO THE HOUSE LOCATED SOUTH OF ROUTE 309 AND OPPOSITE OF HEDGE HOG LANE IS ESTIMATED TO BE 13' (SECTION BB), THE FIRST FLOOR OF THIS HOUSE IS OBSERVED TO BE 11.5± FT. AND THEREFORE THE BASEMENT OF THE HOUSE WOULD BE COMPLETELY FLOODED AS WELL AS THE 1ST FLOOR BY 1.5± FT. OF WATER. THE 1ST FLOOR ELEVATIONS OF OTHER HOUSES ADJACENT TO THE BROOK ARE SUBSTANTIALLY HIGHER THAN THE ESTIMATED FLOOD STAGE. FOR EXAMPLE, THE 1ST FLOOR ELEVATIONS OF THE TWO HOUSES SOUTH EAST OF SECTION CC ARE 12± FT. WHEREAS THE FLOOD STAGE IS ESTIMATED TO BE 8.5± FT. HOWEVER, THE TWO CULVERTS BETWEEN SECTION BB AND SECTION CC ARE EXPECTED TO BE SEVERELY IMPACTED BECAUSE OF HIGH FLOOD STAGE AND HIGH VELOCITIES WHICH ARE IN THE RANGE OF 9 TO 13 FPS.

b) THE CULVERT ON ROUTE 309 COULD STAY — FOR THE OBSERVED CONDITIONS, ONLY 900 ± CFS COULD PASS THROUGH THIS CULVERT. THIS WATER IS ESTIMATED TO HAVE 5 ± FT. DEPTH IN THE VICINITY OF THE HOUSE LOCATED SOUTH OF ROUTE 309 AND OPPOSITE OF HEDGEHOG LANE AND THEREFORE WOULD NOT FLOOD THIS HOUSE, BUT THE BROOK WOULD INUNDATE A PORTION OF WEST MOUNTAIN ROAD.

HOWEVER, AT ROUTE 309 CULVERT, FLOOD WATER WITH 10FT DEPTH IS EXPECTED TO BE 1 ± FT HIGHER THAN THE ROAD SURFACE. THE FIELD OBSERVATIONS INDICATE A POTENTIAL FOR FLOOD WATER TO TRAVEL ON ROUTE 309 TOWARDS HEDGEHOG LANE AND IMPACT TWO HOUSES LOCATED IN THE CORNER OF ROUTE 309 AND HEDGEHOG LANE.

THUS, IT CAN BE SAEN FROM THE ABOVE DISCUSSION THAT IN EITHER OF THE TWO SITUATIONS VISUALIZED, SIGNIFICANT HAZARD IS CONSIDERED LIKELY.

SUMMARY- HYDRAULIC/HYDROLOGIC COMPUTATIONS

TEST FLOOD PEAK INFLOW PMF 1750CFS
(Parallel computations have been performed for ½ PMF peak inflow and results are summarized below)

PERFORMANCE AT PEAK FLOOD CONDITIONS:	PMF	½ PMF
PEAK INFLOWS CFS	1750	900
PEAK OUTFLOWS CFS	1630	785
SPILL.CAP.TO TOP OF DAM (EL.453.8 NGVD)	415	415
SPILL.CAP. TO TOP OF DAM % OF PEAK OUTFLOW	25	53
SPILL.CAP. TO PEAK FLOOD ELEVN.	670	510
SPILL. CAP. TO PEAK FLOOD ELEVN. % OF PEAK OUTFLOW	40	65
PERFORMANCE:		
MAXIMUM POOL ELEVATION NGVD	455.24	454.36
MAXIMUM SURCHARGE HEIGHT ABOVE SPILL.CREST FT.	5.24	4.36
NON-OVERFLOW SECTION OF THE DAM OVERTOPPED FT.	1.44	0.56

DOWNSTREAM FAILURE CONDITIONS:
PEAK FAILURE OUTFLOW CFS 26,200
FLOOD DEPTH IMMEDIATELY D/S FROM DAM 20 FT
CONDITIONS AT INITIAL IMPACT AREA; (FOR RT.309 CULVERT WASHED OUT CONDITION)
ESTIMATED STAGE BEFORE FAILURE WITH 476*CFS 334.3NGVD
ESTIMATED STAGE AFTER FAILURE WITH REACH OUTFLOW 11250CFS 343 NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE ΔY_1 8.7±FT
CONDITIONS AT SECONDARY IMPACT AREA:
SEE FAILURE ANALYSIS DISCUSSION SHEETS 20 & 21 OF H & H COMPUTATIONS FOR CONDITION WHERE RT.309 CULVERT STAYS AS WELL AS FOR SECONDARY IMPACTS

*Includes spillway and 18" outlet discharges

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

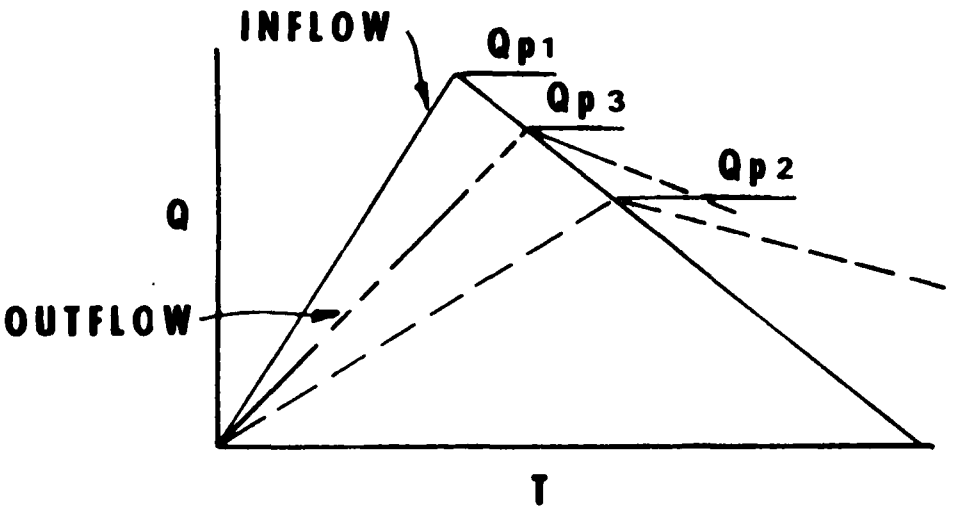
MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS BASED ON TWICE THE STANDARD PROJECT FLOOD (Flat and Coastal Areas)

River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1. Pawtuxet River	19,000	200	190
2. Mill River (R.I.)	8,500	34	500
3. Peters River (R.I.)	3,200	13	490
4. Kettle Brook	8,000	30	530
5. Sudbury River.	11,700	86	270
6. Indian Brook (Hopk.)	1,000	5.9	340
7. Charles River.	6,000	184	65
8. Blackstone River.	43,000	416	200
9. Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Qp_1) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Qp_1".

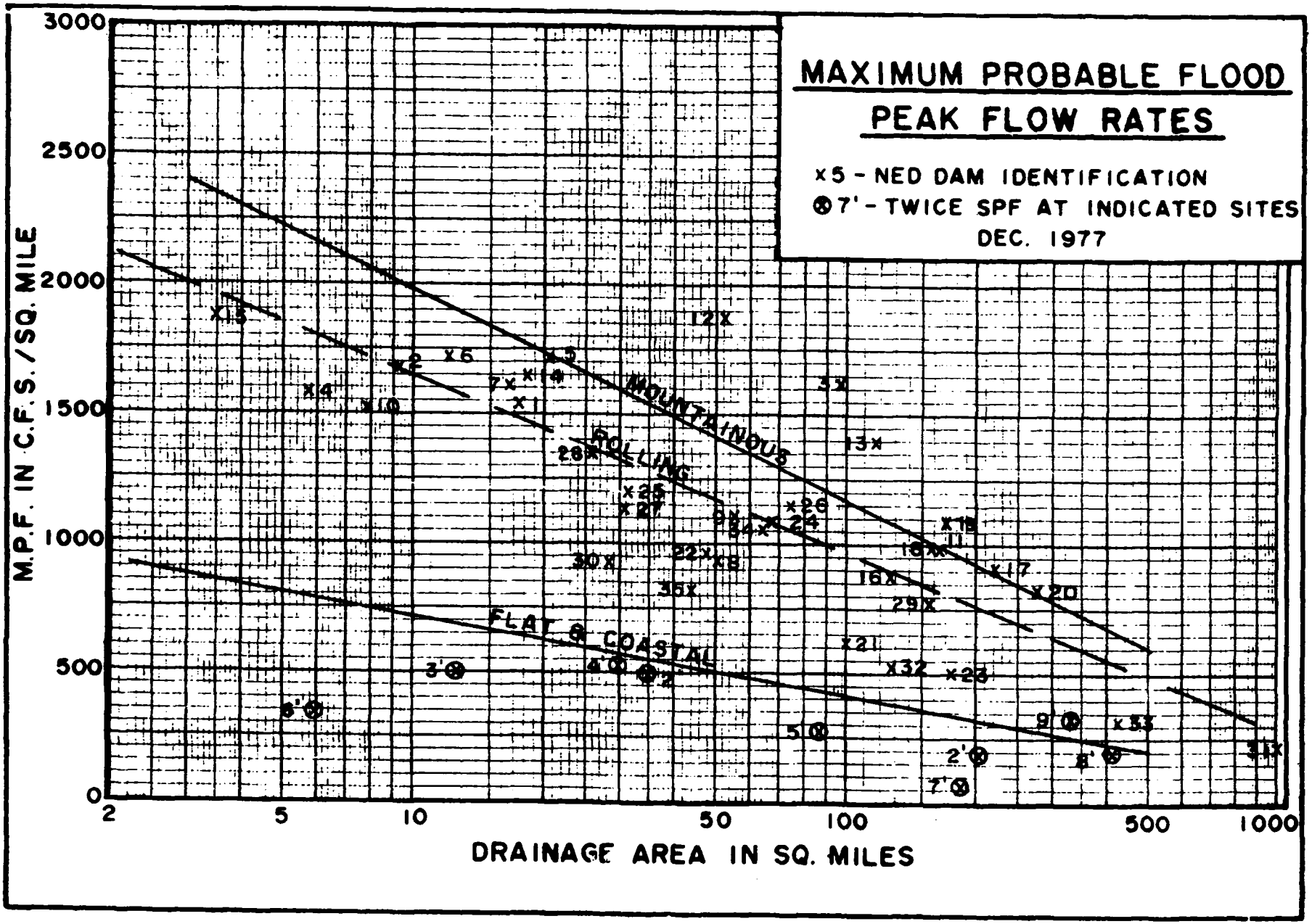
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Qp_2 = Qp_1 \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Qp_3".


MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000
MOUNTAINOUS
ROLLING
FLAT & COASTAL

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

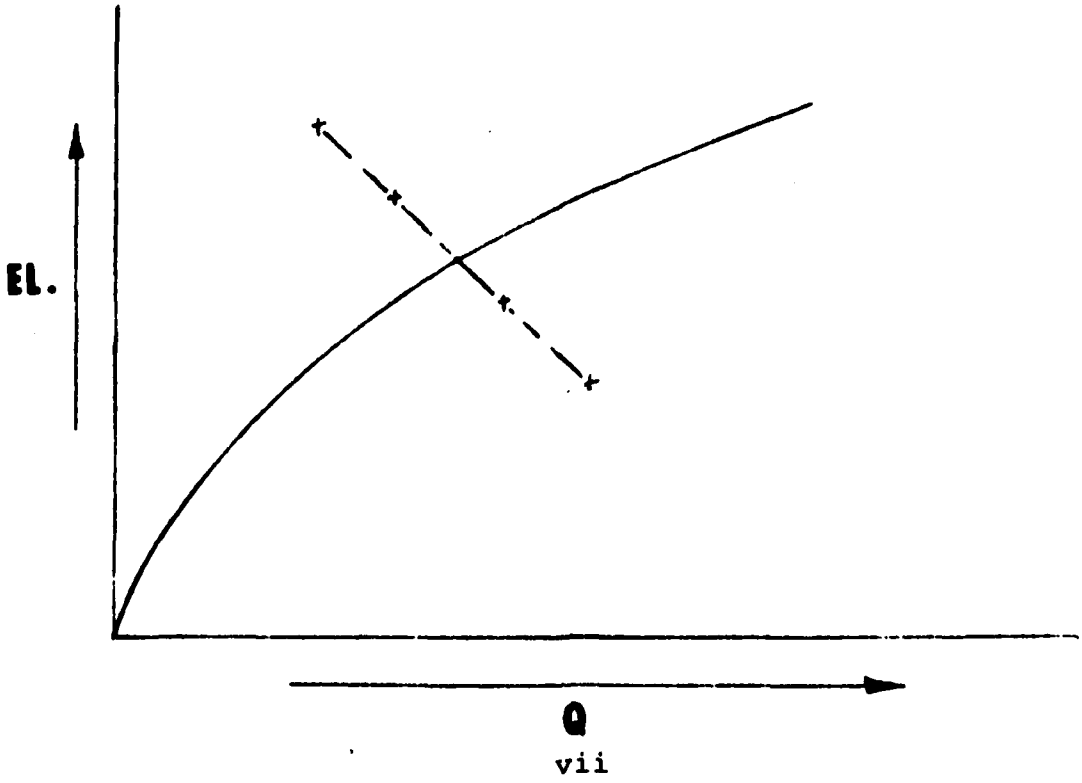
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

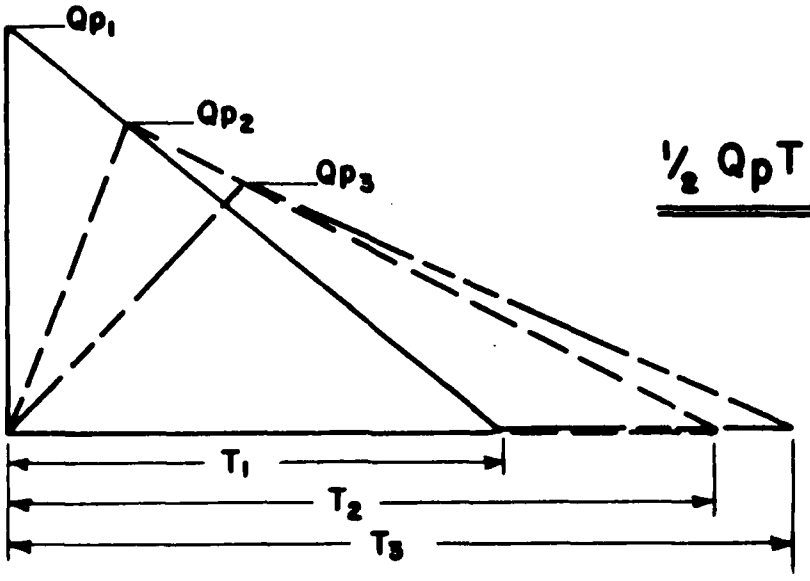
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.
=====	=====	=====



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$\frac{1}{2} Q_p T = 12 S$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g} Y_o^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{ave}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

NOT AVAILABLE AT THIS TIME

END

FILMED